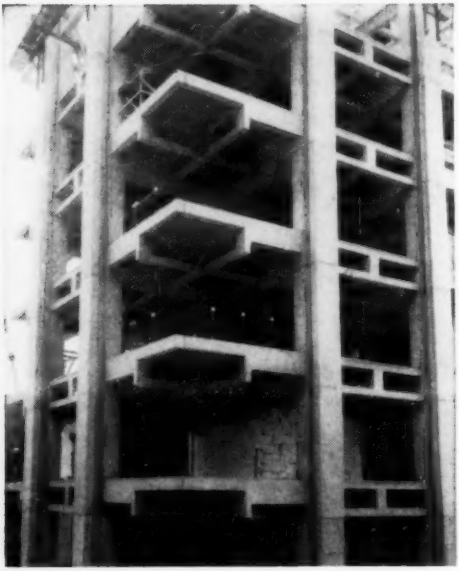
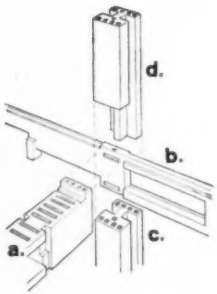
September, 1960
No. 3
V. 32

JOURNAL
OF THE
AMERICAN CONCRETE INSTITUTE

ACI
PROCEEDINGS
V. 57

Bold Adventure in Precast





Example of the complex interlocking. Vierendeel truss (a) and spandrel (b) rest on inside flange of H-column segment (c), with notched end of next H-segment (d) completing the assembly.

● The structural frame of Louis I. Kahn's new medical research building at the University of Pennsylvania is composed of more than a thousand precast members, used with boldness and imagination.

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Publication office; P.O. Box 324, Shepherdsville, Ky. Entered in the Post Office at Shepherdsville, Ky., as mail of the second class.

Editorial and advertising departments at ACI headquarters, P.O. Box 4754, Redford Station, Detroit 19, Mich.

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Published by the American Concrete Institute under the authority of its Board of Direction

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Coming next month in the JOURNAL

Six papers on hyperbolic paraboloids will appear in the October JOURNAL. Two of the papers are devoted to research, two to design, and two to construction.

FELIX CANDELA opens the series with his design paper on "General Formulas for Membrane Stresses in Hyperbolic Paraboloidical Shells."

The construction phase is discussed by GORDON MADSEN and DUTTON BIGGS in "Building for Economy with the Hyperbolic Paraboloid."

HOWARD P. HARRENSTIEN contributes his structural research paper on "Hyperbolic-Paraboloidal Umbrella Shells Under Vertical Loads."

ANTON TEDESKO covers design-construction considerations in "Shell at Denver—Hyperbolic Paraboloidal Structure of Wide Span."

"Experiments with Thin-Shell Structural Models" by J. L. WALING and LONGIN B. GRESCZUK furnishes further structural research data.

L. FISHER gives further design details in "Determination of Membrane Stresses in an Elliptic Paraboloid by Means of Polynomials."

J. TAUB and A. M. NEVILLE offer Part 3 of a five-part series on "The Resistance to Shear of Reinforced Concrete Beams. Part 3—Beams with Bent-Up Bars."

On the Cover—Number One Davis (Medical Office Building), Davis Islands, Tampa, Fla. This eye-appealing building is of reinforced lightweight structural concrete with exposed aggregate precast wall panels and precast sunscreens and sunshades. For a detailed description of the project, see p. 4 of News Letter.—Photo courtesy Georgia Lightweight Aggregate Co.

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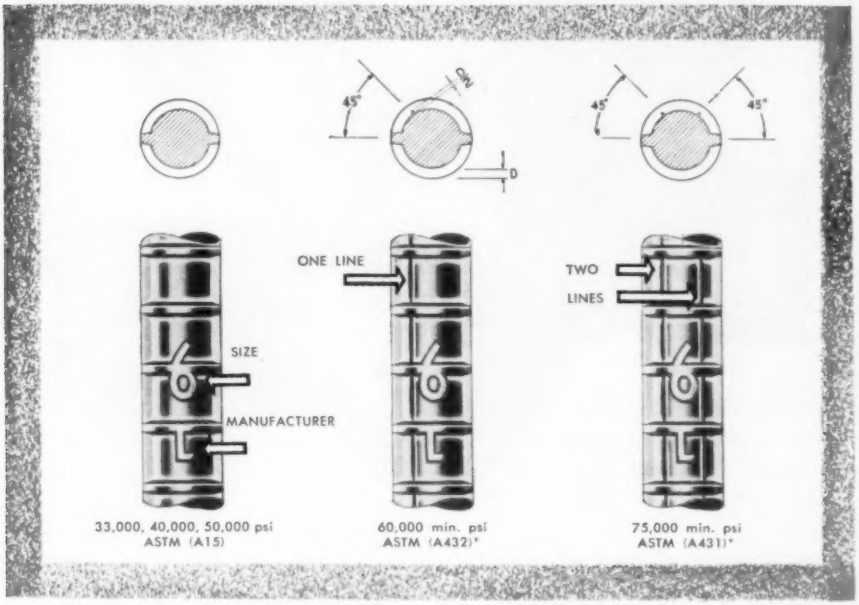

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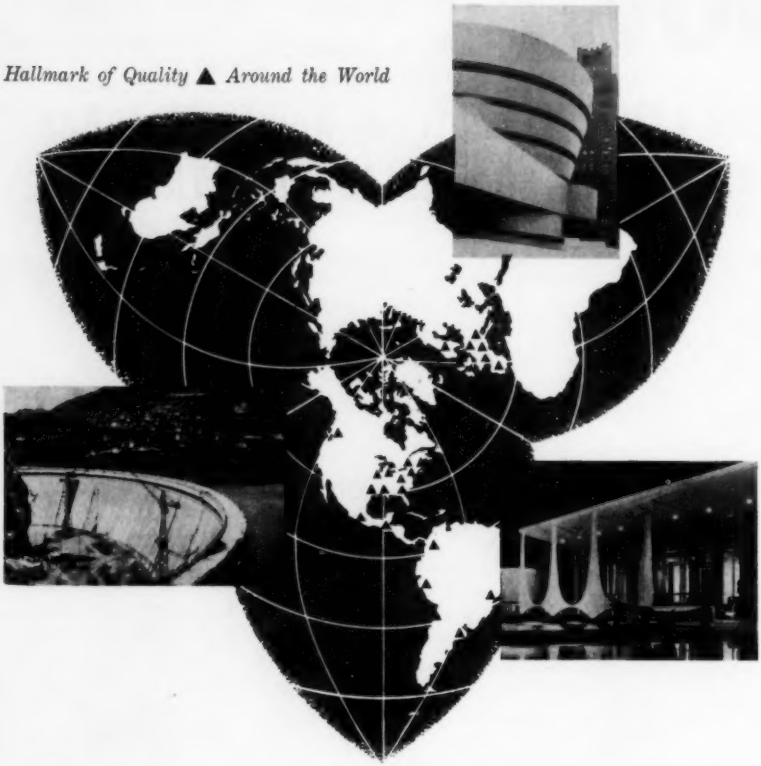

Progress in modern concrete technology has given architecture wings. Dramatic new shapes brighten the structural face of the free world.

Partner in this progress, and notable in its achievements, has been Kaspar Winkler & Co. of Switzerland, this year celebrating its 50th anniversary. Its researchers have developed more than a score of products to modify effectively the characteristics of concrete. Sika Chemical Corporation and the 14 other independent manufacturing companies, located on three continents, proudly salute the accomplishments of the first Sika Company on its anniversary.

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Final report of a project sponsored by Reinforced Concrete Research Council at Cornell University

High-Strength Deformed Steel Bars for Concrete Reinforcement

By SIDNEY A. GURALNICK

Results obtained from the testing to destruction of 42 T-beams reinforced with metallurgically-produced high-strength steel deformed bars are presented. Both flexural and shearing failures occurred and the results are compared with the ultimate strength design procedure given in the appendix of the 1956 ACI Building Code and certain other design procedures. Deflections of test beams are compared with values obtained by use of existing methods for computing deflections. Crack data is also reported and evaluated.

■ HIGH-STRENGTH STEEL AS REINFORCEMENT for concrete received some serious attention by researchers and by the construction industry as long ago as the early 1900's. Recent developments abroad, notably in Sweden, Germany, and Austria, have caused renewed interest in the use of this material in concrete structures.

The term "high-strength" is somewhat ambiguous since it usually has been employed to describe the type of steel used in cables and strands for prestressed concrete or suspension bridge structures. Such steels may have proof stresses of the order of 200,000 psi. However, in the case of ordinary reinforced concrete structures the term "high-strength" steel is used to designate steels having a yield stress in excess of 50,000 psi. In this paper the latter definition will be used.

There are two general methods for producing a high-strength steel. One is by metallurgical means in which relatively high carbon content steel is alloyed with small amounts of such elements as: silicon, phosphorous, nickel, chromium, manganese, or molybdenum. The other method of obtaining a high-strength steel is by cold working an ordinary grade of steel. The usual methods of cold working reinforcing bars are by cold stretching or by cold twisting between two, longitudinally-constrained chucks. The cold-twisted bar is at present the more common of the two types of cold-worked bars.

ACI member **Sidney A. Guralnick** is assistant professor of civil engineering at Illinois Institute of Technology, Chicago. His duties combine teaching and research in the fields of structural theory, reinforced concrete, and thin shell construction. Previously, Dr. Guralnick had engaged in research in the fields of deflection and shear strength of reinforced concrete beams while serving as manager of the Structural Research Laboratory at Cornell University where the research work reported in this paper was conducted.

A metallurgically-produced, high-strength steel or "alloy" steel differs from a cold-worked steel in several important particulars. First, alloy steels can be produced which have stress-strain curves with sharp, definite yield points whereas cold-worked steels are almost always of the gradual-yielding type. Second, alloy steels can be produced which maintain the customary relation of yield stress to ultimate stress (i.e., yield stress approximately 60 percent of ultimate stress); whereas, cold working a steel simply increases the ratio of yield stress to ultimate stress without affecting the ultimate stress significantly. Lastly, some alloy steels have lower ductilities than those exhibited by the steel in cold-stretched or cold-twisted reinforcing bar thus making such alloy steel bars relatively more difficult to bend.

Considerable information is available, particularly in the foreign literature, on test behavior of structural members reinforced with cold-worked, high-strength steel bars but comparatively little test data are available on members reinforced with metallurgically-produced high-strength steel. In view of the differences between the two types of high-strength bars and attracted by the prospect of the sizeable economies apparently possible with high-strength reinforcement, this investigation was initiated and carried out at Cornell University utilizing metallurgically-produced, high-strength bars as concrete reinforcement.

OBJECT AND SCOPE

The object of the experimental investigation reported herein was to provide information on the flexural strength, shear strength, load-deflection, and cracking characteristics of concrete T-beams reinforced with high-strength, alloy steel, deformed bars. The beam specimens were chosen to simulate in the laboratory actual construction conditions as realistically as possible without sacrificing necessary scientific precision. Thirty-four restrained T-beams 17 to 20 ft long were tested to destruction. Twenty-two of these beams were loaded by concentrated forces, and the remaining 12 were loaded by a series of closely spaced hydraulic rams to simulate the effect of a uniformly distributed load. Eight beams were retested over their undamaged portions with two, symmetrically placed, concentrated loads on a simple span of 8 ft, making a total of 42 beam tests. All dimensions and other details of the beams are shown on Fig. 1a-1e. Two concrete strengths were used having nominal values of 3000 and 5000 psi.

All longitudinal reinforcement conformed to ASTM A 305 for deformations and was of alloy steel having a nominal yield strength of 80,000 psi. Beams with plain webs as well as beams having vertical stirrups were tested. Experimental data included crack width measurements, beam deflections, strains in web steel, strains in longitudinal steel, and bar end slip.

Notation

A_s = area of tension reinforcement
A_s' = area of compression reinforcement
a = shear span or distance from plane of nearest concentrated load to plane of support
b = over-all width of flange of T-beam
b' = width of web of T-beam
C = total internal compressive force in the concrete
c = length of arm of internal couple of flexural resisting moment, or distance between C and T
d = effective depth, or distance between outermost compression fiber of beam and the centroid of the tension reinforcement
d' = distance between outermost compression fiber of beam and the centroid of the compression reinforcement
E_c = secant modulus of elasticity of concrete*
E_s = modulus of elasticity of steel*
E_t = initial tangent modulus of concrete*
f_c' = cylinder compressive strength of concrete
f_c = compressive flexural stress in concrete
f_r = tensile modulus of rupture for concrete
f_s = stress in longitudinal reinforcing steel
f_v = stress in vertical stirrups*
f_y = yield stress of reinforcing steel
I = moment of inertia
I_g = gross moment of inertia of whole concrete cross section
I_e = effective moment of inertia of a reinforced concrete beam
I_r = moment of inertia of cracked or "transformed" rectangular cross section
I_t = moment of inertia of cracked or "transformed" T cross section
h = over-all depth of beam section
k = ratio of depth of neutral axis to effective depth
k_1, k_2, k_3 = parameters describing shape and size of flexural compression stress block
L = total length of beam
M = bending moment
M_f = calculated ultimate flexural resisting moment
M_{fs} = calculated ultimate flexural resisting moment assuming that steel stress is limited to 60,000 psi and concrete stress to 0.85 f_c'
M_{sl} = "service-load" moment—$M_{fs}/1.8$
M_u = maximum applied moment at failure in test beam*
n = E_s/E_c, the modular ratio
p = longitudinal tension steel percentage as defined in the text
p' = longitudinal compression steel percentage as defined in the text
Q_g = first statical moment of gross concrete area above neutral axis about same
r = web steel percentage referred to the stem of the T-section
s = spacing of vertical stirrups
T = total force in the tensile reinforcement

*In these cases a prime (′) or double prime (″) mark over a letter indicates that the quantity so designated is the result of calculation, not test measurement. Also, a bar (-) over the letter indicates the quantity contains the dead load as well as the applied load.

t = thickness of flange of T-beam
V = shear force acting on a given cross section of a beam
V_c = resisting shear force supplied by the compression concrete
V_{cr} = shear at which the first diagonal tension crack forms*
V_f = predicted shear at failure in a beam expected to fail in flexure
V_s = resisting shear force supplied by the web reinforcement
V_{tot} = predicted shear at failure in a beam expected to fail in shear (i.e., in a shear-compression type of failure)
V_u = measured maximum shear force occurring in test beam at failure†
V' = predicted shear at failure in a beam test
V_{ui} = maximum shearing force allowable in beam under the provisions of Chapter 8, ACI Building Code (ACI 318-56)
v = shear stress acting on a given plane of a stressed element
v'_{cr} = predicted shear stress at failure of a material subjected to a "pure shear" state of stress
δ = actual deflection of test beam measured at a point just below the interior load in the case of restrained beams, or at the beam center line in the case of simple beams*

EXPERIMENTAL PROGRAM

The test specimens were all T-beams with identical cross sections. Most of the beams were designed to fail in shear and the remainder were designed to fail in flexure. All longitudinal bars in the test specimens were sufficiently well anchored by embedment to prevent premature bond failures. Thirty-four of the beams‡ were provided with a loaded overhang at one end to produce a condition of high shear and high moment over a support such as would occur at an interior support of a multispan, continuous beam. Eight of the restrained beams were retested over their undamaged portions as simply supported beams making a total of 42 beam tests.

Details of the specimens and description of materials and testing techniques are given in the appendix. Properties of the beam specimens are given in Table 1 and shown in Fig. 1a-1e.

TEST RESULTS

Results of tests on all beam specimens are summarized in Table 2. This table gives the maximum shear at the formation of the first diagonal tension crack; the maximum shear, maximum positive and negative moments at failure; the type of failure, as well as deflection, maximum tensile steel strain, and maximum compression steel strain at a load just prior to failure.

Diagonal tension cracking was assessed by visual observation. The diagonal tension cracking load was taken as the load at which a crack,

*In these cases a prime (′) or double prime (″) mark over a letter indicates that the quantity so designated is the result of calculation, not test measurement.

†A bar (-) over the letter indicates the quantity contains the dead load as well as the applied load.

‡These beams are called "restrained" beams in this paper.

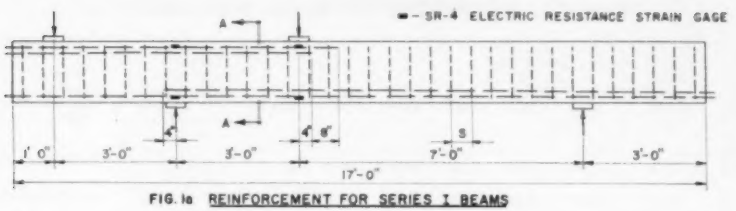


FIG. 1a REINFORCEMENT FOR SERIES I BEAMS

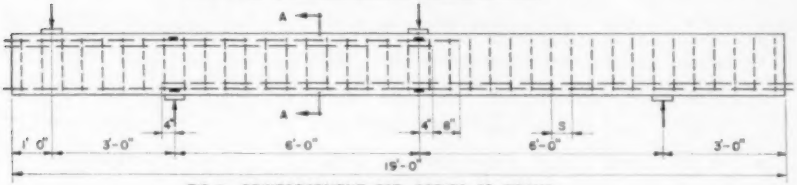


FIG. 1b REINFORCEMENT FOR SERIES II BEAMS

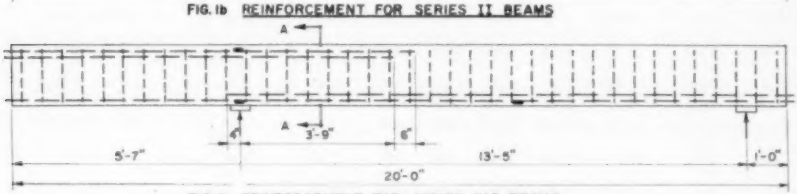


FIG. 1c REINFORCEMENT FOR SERIES III BEAMS

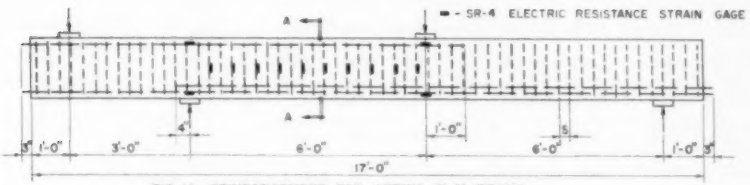


FIG. 1d REINFORCEMENT FOR SERIES II-M BEAMS

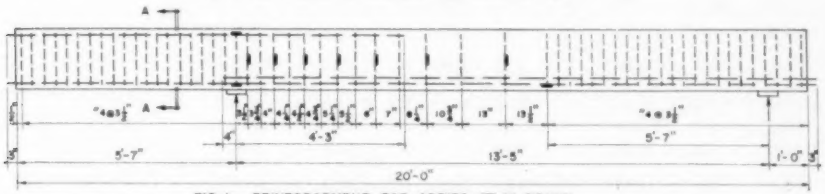


FIG. 1e REINFORCEMENT FOR SERIES III-M BEAMS

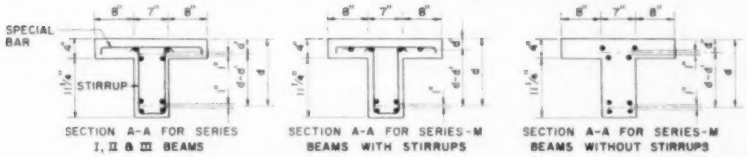

SPECIAL BAR
STIRRUP
8" 7" 8"
8" 7" 8"
8" 7" 8"
SECTION A-A FOR SERIES I, II & III BEAMS
SECTION A-A FOR SERIES-M BEAMS WITH STIRRUPS
SECTION A-A FOR SERIES-M BEAMS WITHOUT STIRRUPS

Fig. 1a-1e—Disposition of reinforcing steel in beams

TABLE 1—PROPERTIES OF BEAM SPECIMENS*

Beam	Length, ft	d, in.	d—d', in.	a/d	f_c', psi	f_r, psi	f_y, ksi	Longitudinal reinforcement, tensile and compressive	Web reinforcement	Age at test, days
IA-1	17.0	11.81	9.07	3.05	3230	315	87.7	2-# 7 2-# 6	#4@3½ in.	29
-2	17.0	12.05	9.24	2.99	2620	330	87.7	2-# 7 2-# 6	#2@5¾ in.	29
IB-1	17.0	11.81	9.50	3.05	2440	310	84.6	2-# 7	#4@3½ in.	30
-2	17.0	12.12	9.49	2.97	2440	310	84.6	2-# 7	#2@5¾ in.	31
IC-1	17.0	11.78	9.10	3.06	4930	470	83.9	2-# 7 2-#10	#4@3½ in.	27
-2	17.0	12.19	9.18	2.95	4930	470	83.9	2-# 7 2-#10	#2@5¾ in.	31
ID-1	17.0	11.81	9.09	3.04	4930	470	87.7	2-# 6 2-# 7	#4@3½ in.	32
-2	17.0	12.05	9.24	2.99	4930	470	87.7	2-# 6 2-# 7	#2@5¾ in.	28
IIA-1	19.0	11.81	9.07	6.10	3230	315	87.7	2-# 7 2-# 6	#4@3½ in.	28
-2	19.0	12.05	9.24	5.98	2620	330	87.7	2-# 7 2-# 6	#2@5¾ in.	30
IIB-1	19.0	11.81	9.50	6.10	2440	310	84.6	2-# 7	#4@3½ in.	29
-2	19.0	12.12	9.49	5.94	2440	310	84.6	2-# 7	#2@5¾ in.	28
IIC-1	19.0	11.78	9.10	6.11	5520	486	83.9	2-# 7 2-#10	#4@3½ in.	31
-2	19.0	12.19	9.18	5.91	5520	486	83.9	2-# 7 2-#10	#2@5¾ in.	29
IID-1	19.0	11.81	9.09	6.10	5520	486	87.7	2-# 6 2-# 7	#4@3½ in.	30
-2	19.0	12.05	9.24	5.98	5520	486	87.7	2-# 6 2-# 7	#2@5¾ in.	28
IIIA-1	20.0	11.81	9.07	—	3130	356	87.7	2-# 7 2-# 6	#4@3½ in.	31
-2	20.0	12.05	9.24	—	3130	356	87.7	2-# 7 2-# 6	#2@5¾ in.	30
IIIB-1	20.0	11.81	9.50	—	3130	356	84.6	2-# 7	#4@3½ in.	30
-2	20.0	12.12	9.49	—	3130	356	84.6	2-# 7	#2@5¾ in.	33
IIIC-1	20.0	11.78	9.10	—	5300	526	83.9	2-# 7 2-#10	#4@3½ in.	41
-2	20.0	12.19	9.18	—	5300	526	83.9	2-# 7 2-#10	#2@5¾ in.	40
IIID-1	20.0	11.81	9.09	—	5300	526	87.7	2-# 6 2-# 7	#4@3½ in.	30
-2	20.0	12.05	9.24	—	5300	526	87.7	2-# 6 2-# 7	#2@5¾ in.	38
IA-1R	8.0	11.81	—	3.05	3230	315	87.7	2-# 7 2-# 6	#4@3½ in.	40
-2R	8.0	12.05	—	2.99	2620	330	87.7	2-# 7 2-# 6	#2@5¾ in.	29
IB-1R	8.0	11.81	—	3.05	2440	310	84.6	2-# 7	#4@3½ in.	33
-2R	8.0	12.12	—	2.97	2440	310	84.6	2-# 7	#2@5¾ in.	33
IC-1R	8.0	11.78	—	3.06	4930	470	83.9	2-# 7 2-#10	#4@3½ in.	28
-2R	8.0	12.19	—	2.95	4930	470	83.9	2-# 7 2-#10	#2@5¾ in.	32
ID-1R	8.0	11.81	—	3.05	4930	470	87.7	2-# 6 2-# 7	#4@3½ in.	34
-2R	8.0	12.05	—	2.99	4930	470	87.7	2-# 6 2-# 7	#2@5¾ in.	28
IA-1M	17.0	12.55	10.09	2.87	3898	390	92.2	2-# 7 2-# 6		31
IC-1M	17.0	12.46	9.92	2.89	4862	455	83.7	2-# 7 2-#10		32
IIA-1M	19.0	12.55	10.09	5.74	3898	390	92.2	2-# 7 2-# 6		28
IIB-1M	17.0	12.63	11.07	5.70	3237	366	103.1	4-# 5	#4@3½ in.	28

*For all beams: $b' = 7.0$ in., $b = 23.0$ in., $t = 4.0$ in., $h = 15.25$ in.

TABLE 1 (Cont.)—PROPERTIES OF BEAM SPECIMENS*

Beam	Length, ft	d, in.	$d-d'$, in.	a/d	f_c', psi	f_r, psi	f_y, ksi	Longitudinal reinforcement, tensile and compressive	Web reinforcement	Age at test, days
IIC-1M	19.0	12.46	9.92	5.78	4862	455	83.7	2-# 7 2-#10		29
IID-1M	17.0	12.55	10.89	5.74	4493	461	92.2	2-# 7 2-# 6		29
IIIA-1M	20.0	12.55	10.09		4019	428	92.2	2-# 7 2-# 6		27
IIIB-1M	20.0	12.63	11.07		3237	366	103.1	4-# 5	#4 var.	28
IIIC-1M	20.0	12.46	9.92		4862	455	83.7	2-# 7 2-#10		28
IIID-1M	20.0	12.55	10.89		4493	461	92.2	2-# 7 2-# 6	#4 var.	27

*For all beams: $b' = 7.0$ in., $b = 23.0$ in., $t = 4.0$ in., $h = 15.25$ in.

inclined at approximately 45 deg to the horizontal, first intersected the level of the middepth of the beam.

Failure load was taken as the maximum load carried by the beam. In all cases, loading was continued beyond the maximum load until the specimen was destroyed.

Modes of failure

Failure occurred in one of three modes: diagonal tension failure (abbreviated DT), shear-compression failure (abbreviated SC), or flexural tension failure (abbreviated FT). These modes are defined for the purposes of evaluating the test data as follows:

1. Diagonal tension failure is a failure characterized by total destruction of the load carrying capacity of the beam through the prolongation of a diagonal tension crack from the bottom to the top of the beam resulting in a sudden rupture.

2. Shear-compression failure is a failure in which the compression zone of the concrete immediately above a diagonal tension crack is destroyed by crushing, resulting in a complete loss of the carrying capacity of the beam.

3. Flexural tension failure is a failure by yielding of the reinforcement in the tension zone at a region of maximum moment after considerable deflections have taken place and before crushing of the compression concrete has occurred. Final destruction of the beam is through crushing of the compression concrete after the tension steel has yielded and usually occurs at a load only slightly higher than that causing the yielding in the tension steel. For beams reinforced with a "gradual-yielding" steel, as was the case in some of the beams of this investigation, the term "yield point" is defined as the stress at which a specified inelastic strain is attained (e.g., as in the 0.2 percent offset method). In beams reinforced with gradual-yielding steel, the beam may continue to accept significant increases in load after the yield stress has been reached in the tension steel; the beam loses its carrying capacity only after the compression concrete fails by crushing.

TABLE 2—BEAM TEST RESULTS

Beam	Diagonal tension cracking, maximum shear, $\overline{V}_{cr}$, kips	Failure			Deformations at load approaching failure load			
		Maximum shear, $\overline{V}_u$, kips	Maximum positive moment, $+\overline{M}_u$, in.-kips	Maximum negative moment, $-\overline{M}_u$, in.-kips	Type failure*	Deflection at point of maximum positive moment, in.	Maximum measured strains: Tension steel, 10^{-6} in./in.	Maximum measured strains: Compression steel, 10^{-6} in./in.†
IA-1	14.7	76.0	1339	1350	DT	0.387	1690	+ 560
-2	14.7	38.0	662	673	DT	0.190	897	+ 158
IB-1	26.3	69.8	1229	1240	DT+FT	0.477	2854	+ 826
-2	22.9	30.3	518	528	DT	0.236	1189	+ 664
IC-1	26.3	96.7	1710	1724	DT	0.328	1250	+1060
-2	32.1	58.2	1019	1030	DT	0.142	615	+ 675
ID-1	26.3	102.4	1807	1818	DT	0.373	2680	+1160
-2	18.7	58.5	1023	1034	DT	0.199	1205	+ 718
IIA-1	8.7	58.0	2046	2039	DT	0.532	2280	− 310
-2	10.7	30.8	1067	1060	DT	0.430	1479	− 121
IIB-1	18.7	44.2	1549	1542	FT	0.879	2805	− 521
-2	10.7	25.4	873	866	DT	0.447	1989	+ 324
IIC-1	26.7	91.2	3241	3234	DT	0.624	2127	+ 670
-2	26.7	40.8	1427	1420	DT	0.242	1400	+1960
IID-1	18.7	71.0	2467	2460	DT+FT	0.663	2660	− 660
-2	18.7	40.5	1420	1413	DT	0.510	1894	+ 511
IIIA-1	23.3	85.7	2171	1708	FT	1.538	3822	− 169
-2	17.5	52.0	1310	1033	SC	0.695	1838	+ 440
IIIB-1	40.8	64.0	1617	1274	FT	0.962	2726	+ 348
-2	17.5	61.7	1557	1227	SC+FT	1.663	2651	+ 678
IIIC-1	33.0	135.3	3439	2700	DT+FT	0.692	2282	+ 355
-2	33.0	61.7	1558	1227	DT	0.446	1033	+ 895
IIID-1	33.0	106.6	2706	2125	FT	0.890	2800	− 105
-2	33.0	74.1	1875	1475	DT	0.976	2700	+ 700
IA-1R		60.1	2102		FT	0.679		
-2R		37.6	1293		DT	0.360		
IB-1R		42.6	1472		FT	0.985		
-2R		31.5	1075		DT	0.442		

TABLE 2 (Cont.)—BEAM TEST RESULTS

Beam	Diagonal tension cracking, maximum shear, V_{cr}, kips	Failure			Deformations at load approaching failure load			
		Maximum shear, V_u, kips	Maximum positive moment, $+M_u$, in.-kips	Maximum negative moment, $-M_u$, in.-kips	Type failure*	Deflection at point of maximum positive moment, in.	Maximum measured strains	
							Tension steel, 10^{-6} in./in.	Compression steel, 10^{-6} in./in.†
IC-1R		92.6	3272		SC+ FT	0.507		
-2R		50.6	1760		SC	0.312		
ID-1R		63.4	2219		FT	0.409		
-2R		48.9	1697		DT	0.464		
IA-1M	21.4	31.4	543	534	DT	0.150	710	+ 690
IC-1M	26.3	34.9	607	618	DT	0.115	400	+ 850
IIA-1M	16.7	19.1	649	642	DT	0.144	890	− 170
IIB-1M	14.7	53.3	1880	1873	FT	0.785	8680	− 980
IIC-1M	22.7	22.7	779	772	DT	0.104	480	− 30
IID-1M	14.7	77.9	2766	2759	FT	0.740	3640	− 200
IIIA-1M	25.3	33.4	835	661	DT	0.372	1120	+ 280
IIIB-1M	25.3	74.1	1874	1474	FT	1.368	6240	− 310
IIIC-1M	33.0	40.8	1023	808	DT	0.241	660	+ 160
IIID-1M	25.3	107.8	2736	2149	FT	1.719	7700	+ 460

*DT = diagonal tension failure; FT = flexural tension failure; SC = shear-compression failure.
†A negative sign (−) in this column indicates compression and a positive one (+) tension.

Sixteen beams of the total 42 beams tested failed exclusively or primarily in flexural tension. Of the remaining 26 beams, two failed in shear-compression and 24 failed in diagonal tension.

Certain general characteristics of the behavior under load of the test beams were common to all specimens. At low loads little or no cracking was evident. As the test loads increased, flexural cracks in the positive and negative moment tension zones appeared and increased in width, length, and number with increasing loads. In most instances, the first appearance of flexural cracks occurred at a load which corresponded to a computed tensile stress in the beam of about the magnitude of the modulus of rupture of the concrete. Diagonal tension cracks usually did not appear until a load of at least twice the initial flexural cracking load was imposed on the beam. The first diagonal tension cracks appeared invariably in the beam

web in the area between the interior support and the point of contraflexure. As loads were increased, the diagonal tension cracks in this area increased in width, length, and number, and new diagonal tension cracks appeared at other high shear locations in the beam.

Beams which failed in diagonal tension

All six of the beams with plain webs and 18 of the beams with web reinforcement failed in diagonal tension. Failure occurred in these beams when one of the large inclined web cracks extended itself from the bottom to the top of the beam progressing entirely through the flange of the T-beam. This type of failure was quite sudden and it occurred, with one exception, at a load about 20-30 percent higher than the load causing the first diagonal tension crack to appear (Table 2). Beam IIC-1M was the exception; in this case the beam failed immediately after diagonal tension cracking appeared. The inclined crack or cracks causing failure were always easily discernible by the unaided eye. These cracks usually started in the web near the interior support at a load of about two to three times that causing flexural cracks to form.

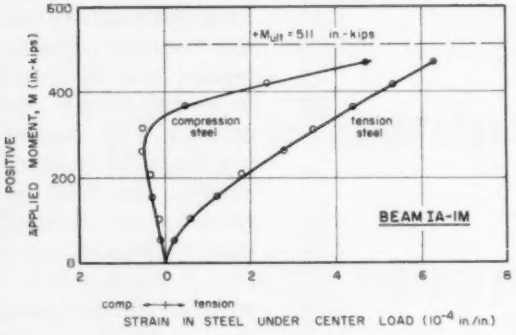
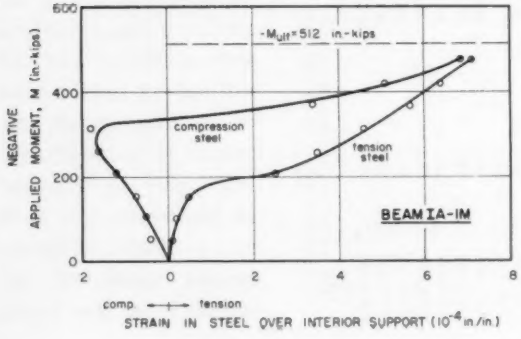


Fig. 2—Typical applied moment versus longitudinal steel strain curves for beams without web reinforcing which failed in diagonal tension



In all cases the diagonal tension cracks formed independently of any existing flexural cracks. In some cases the crack causing failure remained relatively straight along its entire length. In other cases this crack remained fairly straight only until it reached the flange. On reaching the flange, it turned to a horizontal plane and ran along the plane of intersection between the flange and web for a short distance, finally turning upward into the flange at a point between the point of contraflexure and the interior load.

The curves of Fig. 2 show a reversal in direction of strain (from compression to tension) in the compression steel occurring at a load of about two-thirds the ultimate load. This strain reversal which was typical for most of the beams usually took place at the same load which caused one or more diagonal tension cracks to intersect the level of the compression reinforcing. Such changes in direction of strain have also been observed in rectangular beams by Moody, et al.[3]

Beams which failed in shear-compression

Two of the 42 beams tested failed in "pure" shear-compression. These were Beams IIIA-2 and IC-2R. In Beam IIIA-2 the inclined web cracks in the area between the interior support and the point of contraflexure extended deep into the compression concrete over the interior support (about 2 in. of compression area was left uncracked in beams having a total depth of 15¼ in.) Failure occurred by crushing of the small portion of compression concrete remaining uncracked at the interior support. In Beam IC-2R a large inclined web crack extended into an area of the compression flange adjacent to one of the concentrated loads leaving only about a 2 in. depth of compression flange uncracked. Failure occurred by crushing of the small portion of compression concrete remaining uncracked just above the head of the large inclined web crack. The location of the compression failure area was almost 1 ft outside of the region of maximum moment.

Diagonal tension cracking and redistribution of stresses followed the same pattern of development as in the case of the beams failing in diagonal tension. Failures in the shear-compression beams were not nearly as sudden as in the case of the diagonal tension beams. Considerable warning of distress through spalling and cracking was evident in the compression concrete at the failure location before rupture occurred.

Beams which failed in flexure

Eleven beams of the test program failed in "pure" flexural tension. Three beams failed in a manner that can only be described as a combination of diagonal tension and flexural tension. Two other beams failed in a combination of shear compression and flexural tension.

All of the flexural failure beams formed diagonal tension cracks at much higher loads than did any of the other beams in the test program.

The diagonal tension cracks which did form were fewer in number, narrower, and shorter than in any of the other beams. Except in the case of the "combination" failure the diagonal tension cracks did not penetrate the compression flange of the T-beam during any stage of loading. However, at loads approaching the ultimate load, several of the flexural tension cracks in the region of maximum moment did penetrate the compression flange to a depth of approximately 2 in. in most of the retest beams which failed in flexure. Failure occurred by crushing of the small portion of compression concrete remaining uncracked in the region of maximum moment. Failure was quite gradual with warning of distress through spalling and cracking in the compression concrete at failure location before rupture occurred.

"Combination" failures

As was mentioned previously, 5 beams failed in a manner which can only be ascribed to a combination of causes rather than to a single cause.

Beams IB-1, IID-1, and IIIC-1 failed in a combination of diagonal tension and flexural tension. In all these cases the stress in the longitudinal tensile steel at failure was at or very close to the yield stress. Also, the predicted ultimate shear strength and flexural strength were almost equal for these beams; thus this type of failure was to be anticipated in the design of the test specimens. During most of their loading history, these beams behaved similarly to beams which failed in "pure" flexural tension; however, final destruction of the beam was by the prolongation of a diagonal tension crack from the bottom to the top of the section causing a sudden rupture. In these beams, it appears plausible to reason that diagonal tension failure was effectively prevented until after the longitudinal steel yielded. In Beam IID-1 a sudden widening of the flexural cracks over the interior support followed immediately by diagonal tension rupture was actually observed. The sudden widening of the flexural cracks is almost certainly due to steel yielding. This phenomenon occurs so quickly that it is only by the merest chance that the investigator will have the exact failure location under direct observation when failure occurs. Although this sequence of yielding followed by diagonal tension rupture was not actually observed in the other two beams, its occurrence was strongly suspected by reason of the similarity of all the other test data to that of Beam IID-1.

Beam IIIB-2 failed in a combination of shear-compression and flexural tension. In this beam, stress in the positive tension steel and the negative tension steel reached the yield stress before failure occurred. During loading this beam behaved in all respects as if it were going to fail in simple flexural tension. However, at failure, final destruction occurred by crushing of the concrete at the head of two large diagonal tension cracks at a region a short distance away from the region of maximum

moment. From the evidence of steel stresses and high ultimate load, it is probably reasonable to conclude that this beam essentially failed in flexure with a secondary shear-compression effect occurring at, but not before, the ultimate load.

Beam IC-1R failed when the compression concrete in the region of maximum moment failed by crushing. Ordinarily this type of failure would be classified as a flexural failure; however, in this case several diagonal tension cracks penetrated into the compression concrete zone between the load points (i.e., the maximum moment region) thus leading one to the conclusion that the failure was due to a combination of shear-compression and flexural effects.

ANALYSIS OF BEAM TEST RESULTS

Diagonal tension cracking and stirrup stress

The shear, V_{cr}, at which the first diagonal tension crack forms is of primary importance in assessing the shear capacity of a beam, since, by definition, a shear failure (i.e., a shear-compression or diagonal tension failure) can only occur after a diagonal tension crack has formed. Attempts have been made previously to devise equations for predicting the shear at which the first diagonal tension crack will form.[3] Such equations have usually been based on the nominal shearing stress, $v = V/bjd$, and the concrete cylinder compression strength, f_c'. In most beams, the load at which the first diagonal tension crack forms is so low that the whole beam is largely uncracked and stresses throughout the beam are at comparatively low levels. For this reason, the writer believes that shear stresses before diagonal tension cracking occurs may be reliably estimated by using the equation from the elementary theory of homogeneous, isotropic elastic beams

$$v = \frac{VQ}{Ib} \qquad (1)$$

To determine the shear at which the first diagonal tension crack will occur, one may argue that this crack will form when the maximum shear stress calculated by Eq. (1) reaches the stress which would cause an element of plain concrete to fail if it were subjected to a "pure shear" state of stress. Unfortunately, the magnitude of this limiting shear stress, which will be denoted v_{cr}, cannot be determined directly because of the difficulty, if not impossibility, of performing a pure shear test on a plain concrete specimen. Using Mohr's rupture theory, it has been shown[6] that one may use as a conservative estimate of v_{cr} the relation:

$$v'_{cr} = \frac{f_r f_c'}{f_r + 2f_c'} \qquad (2)$$

Using Eq. (1) and (2), the following simple formula for predicting the shear, V'_{cr}, at which the first diagonal tension crack will form may be obtained:

$$V'_{cr} = \frac{I_g b}{Q_g} v'_{cr}$$

$$V'_{cr} = \frac{I_g b}{Q} \times \frac{f_r f_c'}{f_r + 2f_c'} \quad \text{..... (3)}$$

In this equation the terms I_g and Q_g refer to the section properties of the gross concrete cross section not including reinforcement.

The test data and the results of calculations based on Eq. (3) for all beams except the retest specimens appear in Table 3. Ordinarily, diagonal tension cracks may be expected to form in the beam at some distance away from the section of maximum shear — the reason being

TABLE 3—COMPARISON OF TEST RESULTS WITH THEORETICAL INITIAL DIAGONAL TENSION CRACK SHEAR

Beam	$\overline{V}_{cr}$,* kips	f_c', psi	f_r, psi	v'_{cr}, psi	V'_{cr}, kips	$\frac{\overline{V}_{cr}}{V'_{cr}}$
IA-1	14.7	3230	315	151	10.9	1.35
-2	14.7	2620	330	155	11.2	1.31
IB-1	26.3	2440	310	146	10.5	2.50
-2	22.9	2440	310	146	10.5	2.18
IC-1	26.3	4930	470	224	16.2	1.62
-2	32.1	4930	470	224	16.2	1.98
ID-1	26.3	4930	470	224	16.2	1.62
-2	18.7	4930	470	224	16.2	1.15
IIA-1	8.7	3230	315	151	10.9	0.80
-2	10.7	2620	330	155	11.2	0.96
IIB-1	18.7	2440	310	146	10.5	1.78
-2	10.7	2440	310	146	10.5	1.02
IIC-1	26.7	5520	486	233	16.8	1.59
-2	26.7	5520	486	233	16.8	1.59
IID-1	18.7	5520	486	233	16.8	1.11
-2	18.7	5520	486	233	16.8	1.11
IIIA-1	21.5	3130	356	168	12.1	1.78
-2	14.7	3130	356	168	12.1	1.21
IIIB-1	35.0	3130	356	168	12.1	2.90
-2	14.7	3130	356	168	12.1	1.21
IIIC-1	38.2	5300	526	251	18.1	1.56
-2	28.2	5300	526	251	18.1	1.56
IIID-1	28.2	5300	526	251	18.1	1.56
-2	28.2	5300	526	251	18.1	1.56
IA-1M	21.4	3898	390	186	13.4	1.67
IC-1M	26.3	4862	455	218	15.7	1.68
IIA-1M	16.7	3898	390	186	13.4	1.25
IIB-1M	14.7	3237	366	173	12.5	1.18
IIC-1M	22.7	4862	455	218	15.7	1.45
IID-1M	14.7	4493	461	220	15.9	0.92
IIIA-1M	21.5	4019	428	203	14.7	1.46
IIIB-1M	21.5	3237	366	173	12.5	1.72
IIIC-1M	28.2	4862	455	218	15.7	1.80
IIID-1M	21.5	4493	461	220	15.9	1.35
					Avg. =	1.51

*For Series III beams, $\overline{V}_{cr}$ was taken at a section d in. from the interior support.

that diagonal tension cracks usually do not form in regions of the beam in the immediate vicinity of a concentrated load or reaction, because of the presence of high local compressive stresses induced by the concentrated force. Based on observations made in this and other investigations, one may expect the location of the first diagonal tension crack to be removed a distance approximately equal to the beam depth, d, away from the support or concentrated load at which the section of maximum shear is located. For all Series III beams, the values of the shear, $\overline{V}_{cr}$, appearing in the second column of Table 3 have been taken at a section d distant from the interior support on the span between supports. Because the shear was constant between interior support and interior load points in all beams subjected to concentrated loads, the maximum shear at the appearance of the first diagonal tension crack and the shear at the expected crack location were identical in all but the Series III beams.

The column headed $\overline{V}_{cr}/V'_{cr}$ in Table 3 shows that agreement between test and theory is best with the Series II beams and worst with the Series I beams. This was to be anticipated because the M/Vd ratio of the Series I beams was so low (approximately 1.5) that they were definitely in the short, deep beam category[2] and Eq. (1) was not applicable. On the other hand, the M/Vd ratios of the Series II beams

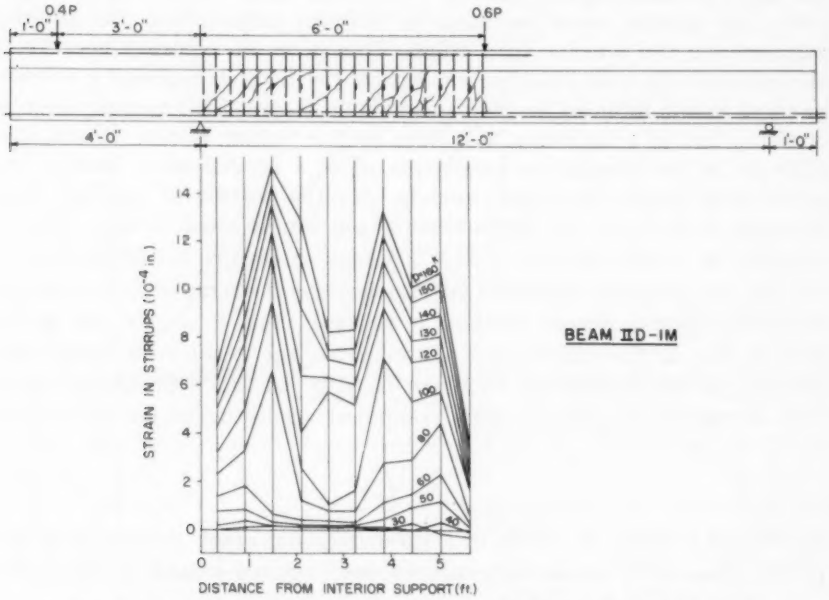


Fig. 3—Stirrup strain distribution at each stage of applied load

were high enough (approximately 3.0) to insure that these beams were not in the short, deep beam category and thus Eq. (1) for computing shear stress was valid. The average value of $\overline{V}_{cr}/V'_{cr}$ for all Series II beams is 1.23 while the average for all the beams in Table 3 is 1.51. It is apparent that in most cases the theory is conservative. Much more work, especially of a statistical nature, remains to be done about the question of initial diagonal tension cracking before any general conclusions can be reached. Tentatively, however, one may conclude from the results of this investigation that for beams in which the M/Vd ratio is 3.0 or larger, Eq. (3) gives a reasonably reliable and conservative estimate of the shear at the appearance of the first diagonal tension crack. In beams in which the M/Vd ratio is less that 3.0, the theory errs significantly on the conservative side.

Strain gages were placed on the stirrups in the high shear regions of Beams II B-1M, II D-1M, III B-1M, and III D-1M. Results of stirrup strain measurements for Beam II D-1M which are representative of those obtained for all four beams provided with stirrup strain gages are shown in Fig. 3. The location of each stirrup and each strain gage as well as the major cracks in the high shear region are also shown. In Fig. 3 one may observe that the crack pattern strongly influences measured stirrup strain. That is, those strain gages directly crossed by a crack record the highest strains, while those gages located fairly far from cracks record the lowest strains. It may also be observed that very low strains were recorded in stirrups adjacent to the interior support and the interior load point. This is to be expected because of the restraint on web cracking caused by the large compressive stresses at these points induced by the concentrated forces. This is experimental confirmation of a suspicion long held by the writer, among others, that stirrups in the immediate neighborhood of a concentrated load or reaction may hardly be called upon to bear the portion of the full shear assigned to them by the application of the simple truss analogy theory.

Since the truss analogy is such a widely accepted design method, a comparison between measured stirrup stress and stirrup stress computed according to this design method is given in Fig. 4. Again, as in the case of Fig. 3, the curves are plotted from the results obtained from the test on the representative Beam IID-1M. In Fig. 4 graphs of measured stress versus stirrup stress computed according to the equation:*

$$f_s' = \frac{sV}{A_v jd} \qquad (4)$$

as well as a graph of width of largest diagonal crack versus shear are given. This latter graph may also be read against a scale of computed

*In this equation V is the total shear force acting on the section under consideration (i.e., the stirrups alone are assumed to carry the total shear).

stirrup stress as well as against actual shear. The 45-deg line appearing in the figure is the graph of the equation:

$$f_v' = f_v$$

The three stirrup stress curves which appear in Fig. 4 are plotted from the data of the stirrup having maximum strains, from the averages of data from the two stirrups having the highest strains, and from the averages of data from all stirrups provided with gages.

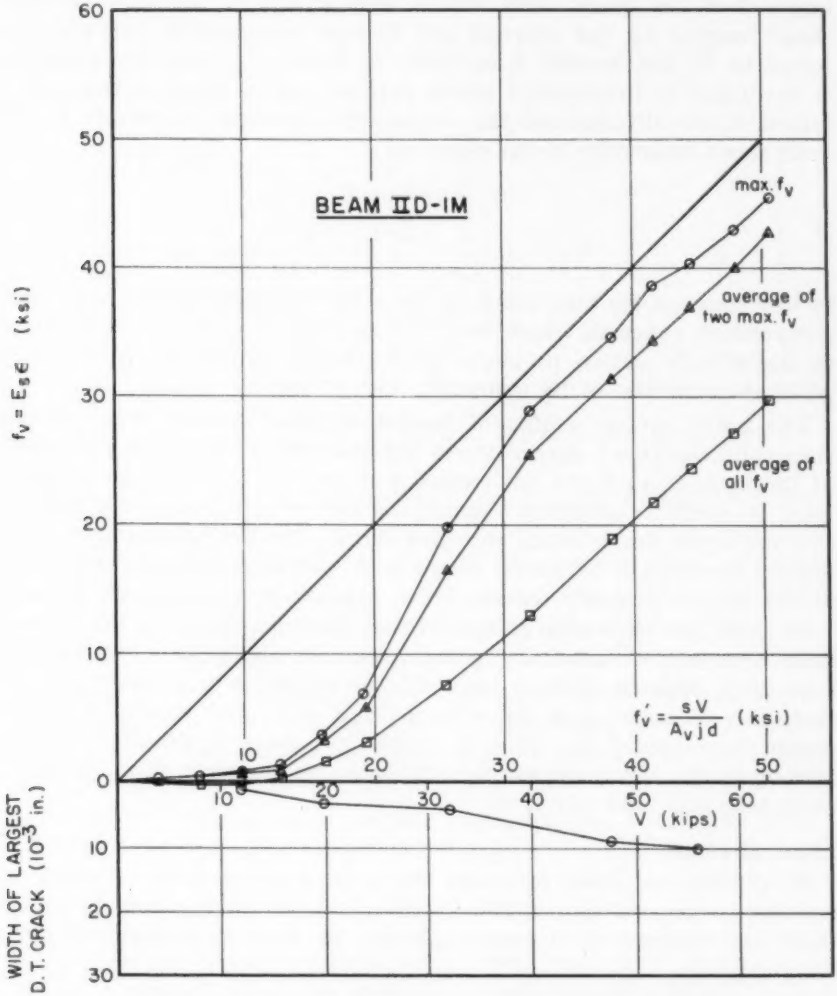


Fig. 4—Measured stirrup stress versus computed stirrup stress curves

In Fig. 4 it may be observed that measured stirrup stresses are very low until the initial diagonal tension cracking load is reached. As soon as this load is imposed on the beam, measured stirrup stresses increase sharply and the f_v versus f_v' curves eventually attain an almost constant slope of approximately 45 deg to the horizontal. The f_v versus f_v' curves all lie below and to the right of the 45-deg line, thus indicating that the stirrups were not carrying the total external shear imposed on the beam (as was assumed in making the f_v' calculation). This means that the uncracked compression concrete also resisted part of the shear imposed on the beam. The relative proportions of the total external shear resisted by the stirrups and by the compressive concrete was found to be nonconstant from beam to beam.* A tentative equation is developed in Reference 7 which permits one to estimate the contribution of the stirrups and the uncracked compression concrete to the total shear resistance of the beam as:

$$V_{tot} = V_c + V_s$$

or

$$V_{tot} = \left(1 + \frac{1 - k_u}{k_u} \times \frac{rf_{vy}}{v_c}\right) V_c \quad \text{.....(5)}$$

In this equation the quantity V_c is the shear resistance of the uncracked compression concrete which is shown in Reference 7 to be a function of the beam's section properties and loading pattern as well as the physical properties of its materials (i.e., f_c' and f_r).

The curve giving widths of largest diagonal tension crack versus shear and computed stirrup stress indicate that in all cases the width of the largest crack did not exceed 0.011 in.† at one-half the ultimate load. As a matter of fact in a number of beams with web reinforcement, the maximum crack width recorded during the entire loading history did not exceed 0.01 in. In all beams with web reinforcement the width of the largest diagonal tension crack appears to vary almost linearly with shear and thus with computed stirrup stress. There is insufficient data, however, to make any general remarks about this phenomenon. One thing appears obvious from this investigation and that is, if all the shear is assigned to the stirrups and computed stirrup stress at design load [using Eq. (4)] is limited to about 20,000 psi, diagonal tension cracks, if any appear, will probably be narrower than the maximum tolerable limit of 0.01 in.

Shear strength

Many semi-empirical equations for predicting the shear strength of reinforced concrete beams have been developed in the past. Some of these are reported in References 3 and 12. Also analytically derived

*See experimental data given by Jones[10] in report of an attempt to measure directly the separate contributions to shear resistance of concrete and web reinforcement.

†A width of 0.01 in. is often used as an upper limit for permissible crack widths in reinforced structures.

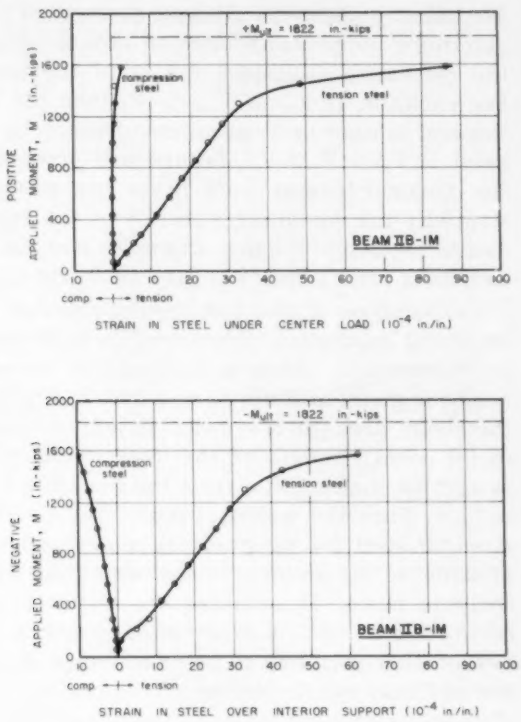


Fig. 5—Typical applied moment versus longitudinal steel strain curves for beams with web reinforcing which failed in flexure

equations based on Mohr's failure theory are given in References 6 and 7. Because of the complexity of many of the empirical expressions developed for predicting shear strength and the present lack of general acceptance of any one of the current approaches to the problem, no quantitative analysis of shear failure is undertaken in this paper.

However, certain assertions of a qualitative nature, based on the test data reported herein and on the analysis given in References 6 and 7, can be made. It has been found that three basic quantities must be determined to carry out a complete analysis of the ultimate strength of a reinforced concrete beam. These are:

1. V_{cr}, the shear at which the first diagonal tension crack forms. This quantity may be computed by means of Eq. (3).

2. V_{tst}, the total shear capacity of a beam expected to fail in shear (i.e., in a diagonal tension or shear-compression type failure). This quantity may be estimated by Eq. (5).

3. V_f, the predicted shear at failure in a beam expected to fail in flexure. This quantity may be computed by use of the equations for predicting the ultimate flexural resisting moment given in References 8, 9, or 14.

To properly assess the ultimate strength of a beam it is first necessary to determine the probable mode of failure. This can be done by comparing the respective computed values of the three quantities given above. For example, if $V_{cr} > V_{tot} > V_f$ then the probable mode of failure is flexural tension or flexural compression as given by V_f. On the other hand, if $V_{cr} > V_f > V_{tot}$ failure will probably again be in flexure since the diagonal tension crack never has an opportunity to form and substantially the whole cross section of the beam resists shear so that V_{tot} has no meaning. Further examples and the derivation of the pertinent equations for V_{cr} and V_{tot} may be found in References 6 and 7.

The analysis of the test results reported herein based on the semi-empirical expressions developed in References 3 and 12 may be found in Reference 5. Also, a theoretically based analysis of the Series II beams is given in References 6 and 7. The latter analysis indicates that the shear strength of a beam expected to fail in shear is independent of the yield strength of the longitudinal reinforcement except insofar as this yield strength affects the expected normal stress distribution at failure. Thus the writer is led to the conclusion that the use of high-strength steel for longitudinal reinforcement poses no additional complication in the problem of computing the shear capacity of a reinforced concrete beam. In addition, the writer believes that the use of such reinforcement will not, in itself, cause a significant shear weakness beyond that normally to be expected in an equivalent beam containing conventional reinforcement.

Flexural strength

The maximum positive moments at failure for all beams which failed in flexure are compared with theoretical ultimate flexural moments in Table 4. Data from strain gages mounted on the compression reinforcement in all beams which failed in flexure indicates that little, if any, stress was being transferred to the compression steel (Fig. 5). Consequently, the effect of compression steel on flexural properties was neglected in all calculations of flexural strength.

The equations used for flexural strength calculations are:

$$M_f = pbd^2 f_y \left(1 - \frac{k_2}{k_1 k_2} \times \frac{p f_y}{f_c'}\right) \quad \text{(6)}$$

$$\frac{k_2}{k_1 k_2} = \frac{1600 + 0.46 f_c' - (f_c')^2/80{,}000}{3900 + 0.35 f_c'} \quad \text{(7)}$$

The latter equation was taken from "Concrete Stress Distribution in Ultimate Strength Design" by Hognestad, Hanson, and McHenry.[8] The flexural strength analysis was based on the section at which maximum positive moment occurred because steel yielding followed by concrete crushing took place at or very close to this section in all beams which

TABLE 4—COMPARISON OF TEST RESULTS WITH THEORETICAL ULTIMATE FLEXURAL STRENGTH VALUES

Beam	Test data		Calculation results			
	Maximum positive moment at failure, $+\bar{M}_u$, in.-kips	Type failure*	$p = \frac{A_s}{bd}$	$\frac{k_3}{k_1 k_2}$	M_f, in.-kips	$\frac{\bar{M}_u}{M_f}$
IB-1	1229	DT + FT	0.44	0.560	1097	1.12
IIB-1	1549	FT	0.44	0.560	1097	1.41
IID-1	2467	DT + FT	0.77	0.645	1984	1.24
IIIA-1	2171	FT	0.77	0.588	1881	1.15
IIIB-1	1617	FT	0.44	0.588	1115	1.45
-2	1557	SC + FT	0.43	0.588	1146	1.36
IIIC-1	3439	DT + FT	1.38	0.642	3170	1.09
IIID-1	2706	FT	0.77	0.642	1978	1.37
IA-1R	2102	FT	0.77	0.590	1889	1.11
IB-1R	1472	FT	0.44	0.560	1097	1.34
IC-1R	3272	SC + FT	1.38	0.638	3134	1.04
ID-1R	2219	FT	0.77	0.638	1966	1.13
IIB-1M	1880	FT	0.43	0.590	1484	1.27
IID-1M	2766	FT	0.72	0.630	2183	1.27
IIIB-1M	1874	FT	0.43	0.590	1484	1.26
IIID-1M	2736	FT	0.72	0.630	2183	1.25
					Avg. =	1.24

*DT = diagonal tension failure; FT = flexural tension failure; and SC = shear-compression failure.

failed in flexure. The ratios of calculated moment, M_f, to actual ultimate moment, M_u, given in Table 4 indicate that the theoretical approach used herein is quite conservative. It should be noticed, however, that the excess flexural strength of test over theory was larger for beams with low percentages of steel than for beams with high percentages of steel. It is believed that the excess flexural strength of these beams is due to the fact that the tensile reinforcing steel had little or no yield plateau and entered the work-hardening range almost immediately upon reaching its (offset) yield stress.* Thus in the case of lightly reinforced beams in which concrete strains are relatively low throughout most of the loading history, the steel may be able to pick up sizeable additional stress after the yield stress is reached before the ultimate concrete strain is reached. Consequently, beam failure occurred by concrete crushing with the steel stress in the work-hardening range at a level considerably above the yield stress. This reasoning is confirmed by strain measurements made on the tension reinforcement in which strains well within the work-hardening range were recorded at beam loads approaching the ultimate (see Table 2). This extra flexural strength of lightly reinforced beams has also been observed by Jensen,[9] and Lash,[11] and may be inferred from the experimental points plotted in Fig. 3 of Whitney's early paper on ultimate strength design.[14] In

*Initial portions of typical stress-strain curves for the reinforcing bar material used in the beam specimens are shown in Fig. A-3.

TABLE 5—COMPARISON OF TEST RESULTS WITH ULTIMATE FLEXURAL STRENGTH VALUES CALCULATED ACCORDING TO APPENDIX, ACI 318-56

Beam	Test data		Calculation results			
	Maximum positive moment at failure, $+\overline{M}_u$, in.-kips	Type failure	M_{f6},* in.-kips	$M_{s1} = \frac{M_{f6}}{1.8}$ in.-kips	$\frac{\overline{M}_u}{M_{f6}}$	$\frac{\overline{M}_u}{M_{s1}}$
IB-1	1229	DT + FT	790	439	1.56	2.80
IIB-1	1549	FT	790	439	1.96	3.53
IID-1	2467	DT + FT	1388	771	1.78	3.20
IIIA-1	2171	FT	1329	738	1.63	2.94
IIIB-1	1617	FT	803	446	2.01	3.63
-2	1557	SC + FT	825	458	1.89	3.40
IIIC-1	3439	DT + FT	2350	1306	1.46	2.63
IIID-1	2706	FT	1384	769	1.96	3.52
IA-1R	2102	FT	1318	732	1.60	2.87
IB-1R	1472	FT	790	439	1.86	3.35
IC-1R	3272	SC + FT	2330	1294	1.40	2.53
ID-1R	2219	FT	1378	766	1.61	2.90
IIB-1M	1880	FT	887	493	2.12	3.82
IID-1M	2766	FT	1463	813	1.89	3.40
IIIB-1M	1874	FT	887	493	2.11	3.80
IIID-1M	2736	FT	1463	813	1.87	3.37
				Avg. =	1.80	3.37

*Based on fictitious yield stress of 60,000 psi.

addition, at the 1957 meeting of the European Committee for Reinforced Concrete, Commission on Steel, under the chairmanship of Prof. Georg Wästlund, it was stated: "For beams with low percentages of such steel the observed failure moments, for short-time tests, are always on the average 5 to 20 percent higher than the moments calculated on the basis of the 0.2 percent offset yield strength." This statement refers to European cold-stretched or cold-twisted steel, which also lacks a yield plateau and goes directly into strain hardening.

A comparison of test results with ultimate moments computed according to the appendix to the ACI Building Code (ACI 318-56) is given in Table 5. The provisions of this part of the code require that the steel stress used in computation of ultimate flexural moment be limited to 60,000 psi and the maximum concrete compressive stress be limited to 0.85 f_c'. The column headed $\overline{M}_u/M_{f6}$ in Table 5 indicates that the ACI Code approach in all cases results in a large underestimation of the actual failure moment. This underestimation, of course, occurs mainly because of the use of the unrealistic value of 60,000 psi for the maximum steel stress at failure.

Load-deflection characteristics

Fig. 6 is a typical moment-deflection curve for one of the beams with web reinforcement which failed in flexure and Fig. 7 is a typical moment-deflection curve for one of the six beams without web reinforcement which failed in shear. A line representing moment versus

deflection based on the gross concrete section and E_t' and one based on the cracked transformed section and E_c' are superimposed on the moment-deflection diagrams of Fig. 6 and 7. It may be observed in both cases typified by these figures that at low loads the test curve of moment versus deflection is quite steep and follows the $E_t'I_g$ line. As the load increases the test curves tend to flatten out and approach and perhaps even cross the $E_c'I_c$ line. It is interesting to observe that the curve of Fig. 7 is really only a blown-up picture of the lower portion of a curve such as the one shown in Fig. 6. Thus the curve of Fig. 7 represents only a small fragment of the full moment-deflection curve the beam might have had if it did not have a weakness in shear and had been allowed to reach its full flexural capacity.

One of the major objectives of this program was to compare actual beam deflections at service loads (as defined by the appendix to ACI 318-56) with values calculated by existing procedures. The "service-load" moment, M_{sl}, is defined herein to be 1/1.8 times the calculated ultimate flexural resisting moment, M_{f6}, assuming that steel stress is limited to 60,000 psi and concrete stress to 0.85 f_c'. The service-load is defined to be the load which produces a maximum moment equal to the service-load moment. The failure moments, $\overline{M}_u$, of those beams which were designed to fail in shear were so low compared to M_{f6} (in some cases $\overline{M}_u$ was even less than M_{sl}), that these beams were unrepresentative of practical beams which are customarily designed to fail in

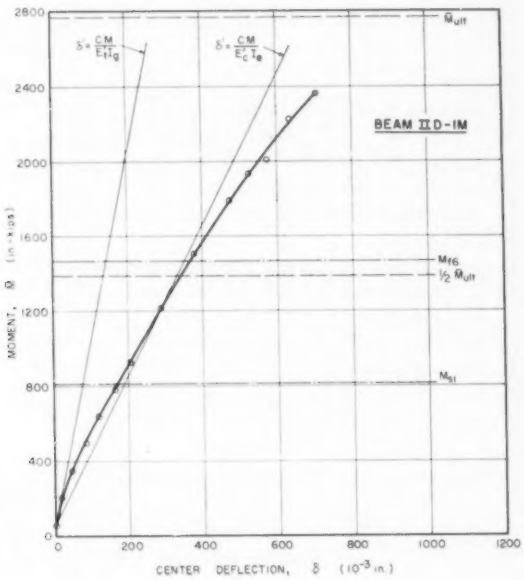


Fig. 6 — Typical moment versus deflection curve for beams with web reinforcing which failed in flexure

TABLE 6—DEFLECTION AT SERVICE-LOAD OF ALL BEAMS WHICH FAILED IN FLEXURE

Beam	$M_{s1} = \frac{M_{ts}}{1.8}$ in.-kips	Test results		Calculated values				$\frac{\delta'}{\delta}$
		Deflection at service-load moment, δ, in.	Type failure	I_{cr},* in.^{4}	I_t,* in.^{4}	$I_e = \frac{I_{cr} + I_t}{2}$ in.^{4}	$E_c' I_e$, 10^6 kip-in.^{2}	Deflection at service-load moment, δ', in.	
IB-1	439	0.150	DT + FT	863	1167	1015	2964	0.127	0.847
IIB-1	439	0.220	FT	863	1167	1015	2964	0.192	0.873
IID-1	771	0.200	DT + FT	956	1318	1137	4935	0.202	1.010
IIIA-1	738	0.390	FT	1151	1656	1404	4549	0.383	0.982
IIIB-1	446	0.480	FT	800	1074	937	3036	0.347	0.723
-2	458	0.330	SC + FT	854	1136	995	3224	0.335	1.015
IIIC-1	1306	0.330	DT + FT	1376	2000	1688	7157	0.431	1.306
IIID-1	769	0.370	FT	972	1344	1158	4910	0.370	1.000
IA-1R	732	0.145	FT	—	1637	1637	5386	0.127	0.876
IB-1R	439	0.120	FT	—	1167	1167	3408	0.121	1.008
IC-1R	1294	0.165	SC + FT	—	2147	2147	8738	0.139	0.842
ID-1R	766	0.125	FT	—	1387	1387	5645	0.127	1.016
IIB-1M	493	0.172	FT	951	1260	1106	3639	0.176	1.023
IID-1M	813	0.170	FT	1189	1649	1419	5492	0.192	1.129
IIIB-1M	493	0.270	FT	951	1260	1106	3639	0.320	1.185
IIID-1M	813	0.270	FT	1189	1649	1419	5492	0.349	1.292
								Avg.	=1.008

*Moment of inertia of cracked or transformed section including tension but not including compression steel.

flexure. Consequently, as suggested to the author by the late Mr. Di Stasio, it appears unwarranted to attempt an analysis of the load-deflection characteristics of such beams. For this reason all further remarks will pertain only to those beams which failed in flexure.

If one of the usual methods such as the double integration, moment-area, or conjugate beam method is used to compute deflections, a question arises as to the value to be used for the "flexural rigidity," EI. Because pronounced flexural cracking had occurred in all beams by the time the service-load was reached, it was concluded that the moment of inertia of the cracked, transformed cross section should be used for I. Furthermore, it was concluded that the secant modulus of concrete at 0.5 f_c' should be used for E since rather high compressive stresses occur at service-loads. For the two types of concrete used in this investigation, it was found that

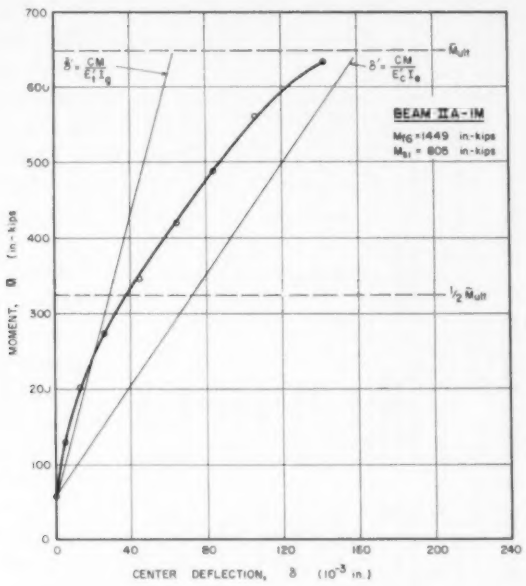


Fig. 7 — Typical moment versus deflection curve for beams without web reinforcing which failed in shear

the secant modulus could be predicted with reasonable accuracy by Inge Lyse's empirical equation: *

$$E_c' = 1.8 \times 10^6 + 460 f_c' \quad \text{.....(8)}$$

The following assumptions served as a basis for the deflection calculations made in this investigation:

a. E is the secant modulus of concrete at 0.5 f_c'.

b. The influence of tension concrete on section properties may be neglected.

c. If the center of gravity of compression reinforcement is very close to the neutral axis of the cross section (as was the case in all beams of this investigation), such reinforcement may be neglected in determining section properties.

d. I is the sum of the moment of inertia of the compression concrete and "n" times the tensile steel area about the neutral axis of the cross section.

e. For continuous or restrained beams in which I of the negative-moment section and I of the positive-moment section do not differ by more than 10 or 15 percent of the larger value, the moment of inertia is constant and is:

$$I_e = \frac{I_r + I_t}{2} \quad \text{.....(9)}$$

in which I_r is the moment of inertia of the negative-moment section and I_t is that for the positive-moment section.†

*Originally proposed for the initial tangent modulus of concrete as Eq. (34) in "A Study of Combined Bending and Axial Load . . .", by Eivind Hognestad, *Bulletin* No. 399, University of Illinois Engineering Experiment Station, Nov. 1951.

†If the beam is a T-beam, I_r is based on a rectangular section and I_t on a T-section.

TABLE 7—CRACK DATA AT SERVICE-LOAD FOR BEAMS WHICH FAILED IN FLEXURE

Beam	M_{fs}, in.-kips	$M_{sl} = \frac{M_{fs}}{1.8}$, in.-kips	Width of cracks				No. of cracks		
			Average of all positive flexural, in.	Average of 2 maximum positive flexural, in.	Average of all negative flexural, in.	Maximum diagonal tension, in.	Positive flexural	Negative flexural	Diagonal tension
IB-1	790	439	0.004	0.008	0.021	0.001	10	2	1
IIB-1	790	439	0.004	0.007	0.015	—	12	2	—
IID-1	1388	771	0.004	0.006	0.021	0.018	14	2	4
IIIA-1	1329	738	0.004	0.010	0.006	0.009	30	4	5
IIIB-1	803	446	0.005	0.009	0.013	—	18	2	—
-2	825	458	0.006	0.014	0.020	0.014	16	1	2
IIIC-1	2350	1306	0.003	0.008	0.008	0.007	31	4	5
IIID-1	1384	769	0.003	0.005	0.014	0.001	29	3	1
IIB-1M	887	493	0.004	0.007	0.007	—	14	6	—
IID-1M	1463	813	0.004	0.007	0.007	0.003	14	7	1
IIIB-1M	887	493	0.004	0.007	0.007	—	16	2	—
IIID-1M	1463	813	0.003	0.005	0.005	0.001	24	5	1

f. The modular ratio, n, equals the modulus of elasticity of the steel, E_s, divided by the secant modulus of the concrete, of E_c', given by Eq. (8).

Deflection calculations, using the product of E and I as the fluexural rigidity term, were carried out by the conjugate beam method for all beams of the investigation which failed in flexure. These calculations were made for a load which imposed a maximum positive moment equal to the service-load moment, M_{sl}, on the beam. The results of these calculations together with pertinent test data appear in Table 6. The extreme right hand column in Table 6 gives the ratios of calculated deflections to measured deflections. A chart of the ratios of calculated deflections to measured deflections is given in Fig. 8.

The grouping of points about the 45-deg line in Fig. 8 indicate rather good agreement between test and theoretical values in all but four cases. In three of these four cases the theory overestimates deflection and is thus on the conservative side. The one instance in which the theory significantly underestimates deflection occurs in the case of Beam IIIB-1. It was observed that the moment-deflection curve for this beam did not follow the trends of other similar beams. It was, therefore, reasoned that Beam IIIB-1 had some unexplained weakness or that there was a defect in its deflection measuring apparatus which was not apparent during the conduct of the test. In any case since there was only one such case of the theory seriously underesti-

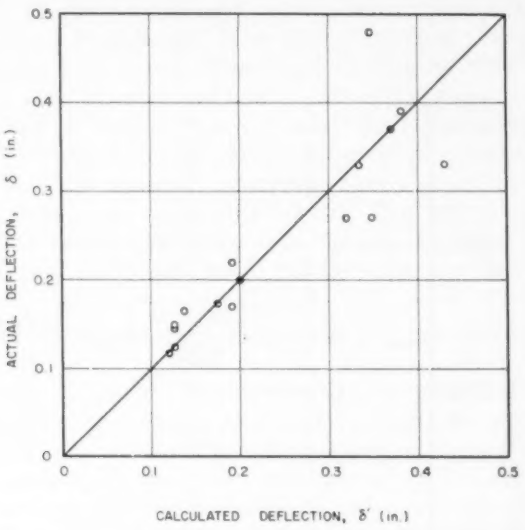


Fig. 8—Comparison of actual deflection with calculated values for all beams failing in flexure

mating the deflection, it may be concluded that the method of calculating short-time static deflections employed herein yields reasonable agreement between test and theory and that significant errors, if any occur, are to be mainly expected on the conservative side. The described method of calculation of deflections is similar to that employed in References 1 and 4.

Flexural crack formation

In testing the Series I, II, and III beams* it was found that negative flexural cracks were few in number but were often wide enough at design loads to be considered objectionable. It was conjectured that this poor negative, flexural crack pattern was a result of the negative tensile reinforcing bars being "bunched" together over the stem of the T-section (see Fig. 1). Consequently, one objective of this program was to investigate the effect on negative flexural cracking of relocating and distributing the negative tensile reinforcement in the flange of the T-beam. Accordingly four beams were tested which were made as nearly identical as possible with the corresponding original-series beams with the exception that the negative tensile reinforcement was distributed across the flange of the T-section (see Fig. 1). These four beams correspond to original-series beams which were designed to fail in flexure and were designated Beams IIB-1M, IID-1M, IIIB-1M, and IID-1M.†

*These beams will hereinafter be called "original-series" beams.
†These beams will hereinafter be called Series-M beams.

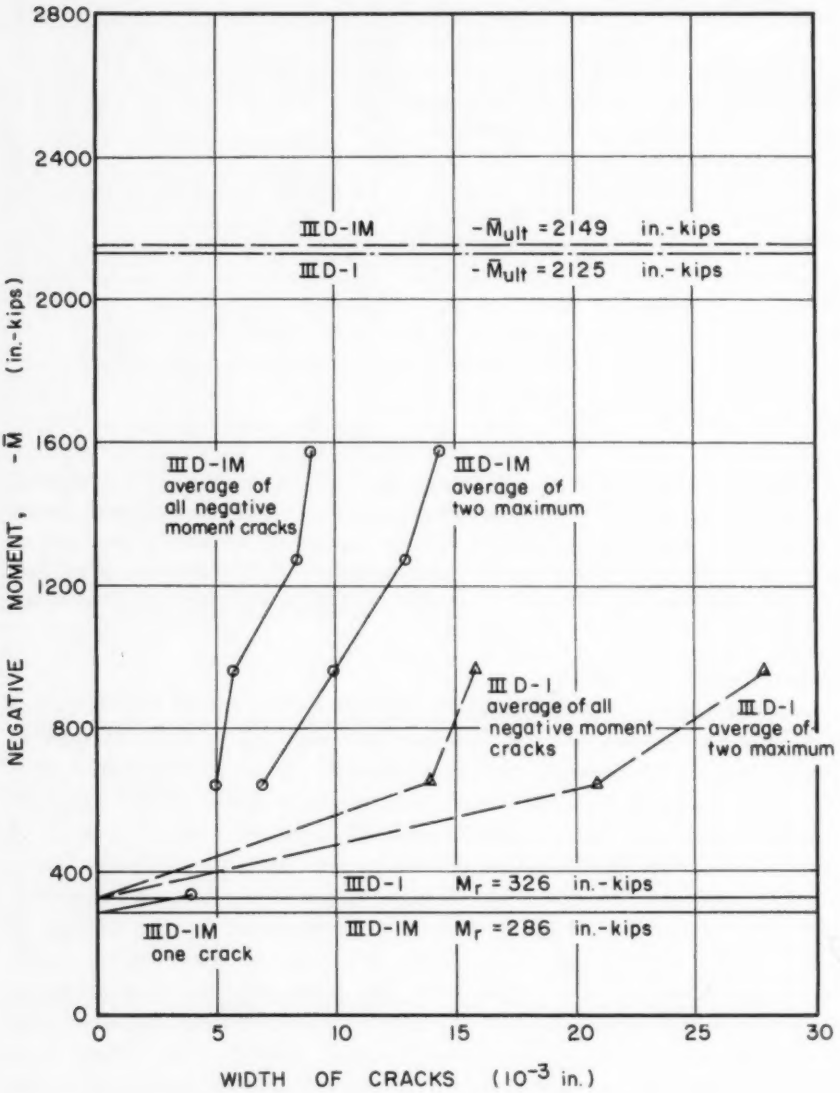


Fig. 9—Comparison of moment versus crack width curves for Beam IIID-1 with bunched negative tension reinforcing and Beam IIID-1M with distributed negative tension reinforcing

Table 7 gives crack data at the service-load for all beams tested in this investigation which failed in flexure. It is quite apparent from this table that negative crack widths at service-loads for the Series M beams represent a considerable improvement over those occurring in the corresponding original-series beams. Fig. 9 gives a set of negative moment versus crack width curves for Beam IIID-1 of the original-series and Beam IIID-1M which is an identical specimen except that the negative reinforcement is distributed across the flange. The improved cracking characteristics of the Series M beam over the corresponding original-series beam are clearly demonstrated in Fig. 9 which is representative of similar comparisons made for all Series M beams with distributed negative tensile reinforcement and corresponding original-series beams.*

Crack widths at service-load for Series M beams varied from approximately one-third to one-half of those in corresponding original-series beams and the average widths at service-loads in no case exceeded a value of 0.007 in. Thus it has been demonstrated that while flexural cracking may be quite pronounced at service-loads in beams reinforced with high-strength steel, it is nevertheless possible to insure adequate crack control by careful attention to proper bar location and placement.

It has been found that flexural crack widths increase in rough proportion to steel stresses; however, the constant of proportionality for the negative flexural cracks was approximately double that for positive flexural cracks. Since in all beams top and bottom steel was identical, this confirms the findings by others that the smaller the ratio of surrounding tensile concrete area to total bar perimeter, the narrower are the widths of flexural cracks.†

Comparison of ultimate loads of beams failing in shear with design loads computed according to Chapter 8 of ACI Building Code

All beams of this project which failed in shear were reviewed according to the provisions of Chapter 8 of the current ACI Building Code (ACI 318-56). The results of these calculations appear in Table 8 along with other data, such as concrete strength, type of failure, and maximum shear at failure. The ratio‡ of the maximum shear at failure, V_u, to the maximum allowable shear, V_{al}, is given for all beams which failed in shear. These "safety factors" range in value from 2.76 to 5.94. The higher values occur in beams having high-strength concrete. This is so because of the cutoff provision which limits the maximum shear stress carried by the concrete to 90 psi. The highest values of the ratio of maximum shear at failure to maximum allowable shear occur in the case of

*Such improvements in crack control obtained by "distributing" reinforcement have also been reported by Wästlund.[13]

†See equations for crack width given by Wästlund.[13]

‡This ratio is designated herein as the "safety factor."

TABLE 8—COMPARISON OF SHEAR STRENGTH OF BEAMS WHICH FAILED IN SHEAR-COMPRESSION OR DIAGONAL TENSION WITH ALLOWABLE SHEAR VALUES

Beam	Longitudinal reinforcement	Web reinforcement	Concrete strength, f_c', psi	Type failure	Maximum shear at failure, $\overline{V}_u$, kips	Maximum allowable shear,* V_{ai}, kips	Safety factor $SF = \frac{\overline{V}_u}{V_{ai}}$
IA-1	2-# 6 2-# 7	#4@3½ in.	3230	DT	76.0	17.36	4.38
-2	2-# 6 2-# 7	#2@5¾ in.	2620	DT	38.0	9.50	4.00
IB-1	2-# 7	#4@3½ in.	2440	DT + FT	69.8	14.12	4.94
-2	2-# 7	#2@5¾ in.	2440	DT	30.3	9.11	3.33
IC-1	2-# 7 2-#10	#4@3½ in.	4930	DT	96.7	17.32	5.58
-2	2-# 7 2-#10	#2@5¾ in.	4930	DT	58.2	10.43	5.58
ID-1	2-# 6 2-# 7	#4@3½ in.	4930	DT	102.4	17.36	5.90
-2	2-# 6 2-# 7	#2@5¾ in.	4930	DT	58.5	10.31	5.67
IIA-1	2-# 6 2-# 7	#4@3½ in.	3230	DT	58.0	17.36	3.34
-2	2-# 6 2-# 7	#2@5¾ in.	2620	DT	30.8	9.50	3.24
IIB-2	2-# 7	#2@5¾ in.	2440	DT	25.4	9.11	2.79
IIC-1	2-# 7 2-#10	#4@3½ in.	5520	DT	91.2	17.32	5.27
-2	2-# 7 2-#10	#2@5¾ in.	5520	DT	40.8	10.43	3.91
IID-1	2-# 6 2-# 7	#4@3½ in.	5520	DT + FT	71.0	17.36	4.09
-2	2-# 6 2-# 7	#2@5¾ in.	5520	DT	40.5	10.31	3.93
IIIA-2	2-# 6 2-# 7	#2@5¾ in.	3130	SC	52.0	10.31	5.04
IIIB-2	2-# 7	#2@5¾ in.	3130	SC + FT	61.7	10.37	5.95
IIIC-1	2-# 7 2-#10	#4@3½ in.	5300	DT + FT	135.3	17.32	7.81
-2	2-# 7 2-#10	#2@5¾ in.	5300	DT	61.7	10.43	5.92
IIID-2	2-# 6 2-# 7	#2@5¾ in.	5300	DT	74.1	10.31	7.19
IA-2R	2-# 6 2-# 7	#2@5¾ in.	2620	DT	37.6	9.50	3.95

TABLE 8 (Cont.)—COMPARISON OF SHEAR STRENGTH OF BEAMS WHICH FAILED IN SHEAR-COMPRESSION OF DIAGONAL TENSION WITH ALLOWABLE SHEAR VALUES

Beam	Longitudinal reinforcement	Web reinforcement	Concrete strength, f'_c, psi	Type failure	Maximum shear at failure, V_u, kips	Maximum allowable shear,* V_{ai}, kips	Safety factor $SF = \frac{V_u}{V_{ai}}$
IB-2R	2-# 7	#2@5¾ in.	2440	DT	31.5	9.11	3.45
IC-2R	2-#7	#2@5¾ in.	4930	SC	50.6	10.43	4.85
	2-#10						
ID-2R	2-#6	#2@5¾ in.	4930	DT	48.9	10.31	4.74
	2-#7						
IA-1M	2-#7	—	3898	DT	31.4	6.92	4.54
	2-#6						
IC-1M	2-#7	—	4862	DT	34.9	6.87	5.08
	2-#10						
IIA-1M	2-#7	—	3898	DT	19.1	6.92	2.76
	2-#6						
IIC-1M	2-#7	—	4862	DT	22.7	6.87	3.30
	2-#10						
IIIA-1M	2-#7	—	4019	DT	33.4	6.92	4.83
	2-#6						
IIIC-1M	2-#7	—	4862	DT	40.8	6.87	4.94
	2-#10						
							Avg. = 4.71

*Maximum allowable shear, V_{ai}, is calculated according to Section 305 and Chapter 8 of ACI Building Code (318-56).

the uniformly loaded beams. The next highest values appear in the case of the Series I beams and the lowest safety factors are evident for the Series II beams. The higher shear strengths of the uniformly loaded beams and the Series I beams are understandable because of their relatively low M/Vd (approximately 1.5) ratios in comparison with those of the Series II beams (M/Vd for Series II beams is approximately 3.0). The former beams were in the short, deep beam category.

In view of these results it appears likely that beams with M/Vd ratios of about 2.0 or less can, in general, be expected to have higher shear strengths than beams with M/Vd ratios which are in excess of 2.0. Since there is no allowance for extra shear strength for beams in the short, deep beam category in the present ACI Building Code, the beams of this program exhibiting such extra strength would, of course, have an extra high safety factor.

CONCLUSIONS

Flexural strength

Of the 42 beams tested, 16 failed exclusively or primarily in flexure. Table 4, which is a comparison of the experimental ultimate moments, $\overline{M}_u$, with the theoretical ultimate moments, M_f, shows that the average value of $\overline{M}/M_f$ is 1.24 and the range 1.04 to 1.45. It is concluded that the excess flexural strength exhibited by the test specimens over that predicted by ultimate strength theory was due to the fact that the tensile reinforcing steel had little or no yield plateau and entered the strain hardening range almost immediately upon reaching its (offset) yield strength. Greater excess flexural strength was exhibited by lightly reinforced beams than by heavily reinforced beams because the concrete strain which occurs simultaneously with the steel yield strain is small thus permitting the steel stress to progress well into the work-hardening range before failure occurs by concrete crushing.

Section A603(e) of the appendix to the 1956 ACI Building Code limits the usable yield strength of high-strength steels to 60,000 psi, whereas the actual yield points of the steels employed in this investigation ranged from 83,700 to 103,100 psi, the majority being between 84,000 and 92,000 psi. If M_{f6} is the ultimate moment computed according to the provisions of the above building code on the basis of a maximum steel stress level of 60,000 psi, the values of $\overline{M}_u/M_{f6}$ ranged from 1.404 to 2.120. Therefore, it is clear that from the standpoint of flexural strength alone, the 60,000 psi limitation on maximum steel stress is overly conservative.

Strain measurements indicated that the compression reinforcement did not contribute significantly to the flexural resistance of the beams tested, regardlless of whether measurements were taken at the maximum positive moment location (i.e., where the flange is in compression and T-beam action occurs) or over the interior support (i.e., where the stem is in compression and the effective cross section is rectangular). It must be concluded that an ultimate theory which is based on the assumption that at failure the stress in the compression reinforcement is at the yield point irrespective of distance of compression steel center-of-gravity, from neutral axis, is not tenable.

Shear strength

Thirty beams of the testing program failed in shear or a combination of shear and flexure. In Table 8 which is a comparison of ultimate shear force, V_u, and the allowable shear force, V_{al}, computed on the basis of Chapter 8 of the 1956 ACI Building Code it is shown that for beams with a concrete strength below 3900 psi the average value of the ratio V_u/V_{al} is 3.98, with a range from 2.76 to 5.95, and for beams

with a concrete strength above 3900 psi the average value of the ratio V_u/V_{al} is 5.21, with a range from 3.30 to 7.81. Several conclusions may be drawn on the basis of this data. First, the use of high-strength steel for beam reinforcement in connection with present building code provisions for shear and diagonal tension results in adequate safety in regard to shear failure. Second, these same shear and diagonal tension code provisions are overly conservative when applied to beams made with concrete having strengths in excess of 3000 psi since a cutoff is imposed at 90 psi, for the maximum allowable shear stress to be carried by the concrete. Lastly, the excessive spread in safety factors which results from the application of present shear and diagonal tension provisions is a further indication that Chapter 8 of the present ACI Building Code is in need of revision.

On the basis of data obtained in this investigation from stirrup strain measurements and certain theories proposed by the author,[6,7] it is concluded that the uncracked compression concrete and the web reinforcement each contribute to the total shear resistance of the beams. No single fixed ratio or coefficient* can serve to relate the contribution of the web reinforcement or the compression concrete to the total shear resistance of the beam. On the contrary, the relative contributions to the total shear resistance are influenced by a number of factors such as beam section properties, loading pattern, and physical properties of beam materials. Furthermore, it has been found that stirrups in the immediate vicinity of concentrated loads or reactions carry a much smaller proportion of the total external shear than elsewhere (for the same value of external shear), due to the presence at such locations of vertical compression stresses which reduce diagonal tension cracking.

Deflections

Following a suggestion made by the late Mr. DiStasio, deflections have been evaluated only for the 16 beams which failed in flexure. This was deemed advisable because the beams which were designed to fail in shear were considered unrepresentative of practical beams which are customarily designed to fail in flexure rather than in shear. Deflections of the test specimens at loads which produced maximum moments equal to the service-load moment were compared with theoretically predicted values. Satisfactory agreement between test results and computed values of deflections were obtained by using values for E computed according to the empirical equation:

$$E_c' = 1.8 \times 10^6 + 460 f_c'$$

and by using the moment of inertia of the cracked, transformed cross section for I. For restrained beams, the arithmetic mean of the moments

*For example, statements such as "at failure, the web reinforcement may be assumed to carry two-thirds of the total external shear . . . ," are made in certain European building codes.

of inertia of the T-section and rectangular section was used as the effective moment of inertia, I_e. For simply supported beams, the effective moment of inertia, I_e, was taken as the moment of inertia of the T-section. On this basis the average ratio of measured deflection to predicted deflection was 1.008 with deviations for 13 of the 16 beams confined to a range of approximately ± 15 percent of the average, the remaining having deviations not exceeding ± 30 percent of the average. It is therefore concluded that the above described method permits a satisfactory estimate to be made of short-time deflections at service-loads.

Flexural cracking

According to the data given in Table 7, the average width of positive moment flexural cracks at service-load (i.e., at a steel stress of the order of 30,000 psi) ranged from 3/1000 to 5/1000 in. with the exception of Beam IIIB-2 which will be discussed separately. The average of the widths of the two widest cracks ranged from 5/1000 to 10/1000 in. These crack widths are not at all excessive if 1/100 in. is regarded as the maximum permissible crack width. Also, it has been found[5] that for beams which failed in flexure, positive moment crack widths were less than 1.5/100 in. at loads up to and including one-half ultimate (corresponding steel stresses being of the order of 50,000 psi). A value of 1.5/100 in. for the maximum crack width may very well be acceptable under low corrosive conditions. The exception to this situation is Beam IIIB-2 which had significantly wider cracks than any of the other beams of Table 7. This beam failed in a combination of shear-compression and flexural tension and was originally designed to fail in shear. Thus it is apparent that a shear weakness may influence flexural crack control deleteriously; or conversely, adequate shear strength may contribute to good crack control.

In Series I, II, and III beams tension reinforcement over the continuous support was bunched within the stem width. In correspondingly numbered Series M beams this reinforcement was spread out uniformly over the width of the flange. Inspection of the relevant data in Table 7 reveals that negative flexural cracks in beams with bunched reinforcement were few in number and excessively wide at service-load. Spreading the reinforcement over the whole flange width resulted in a much larger number of negative flexural cracks as well as a reduction of the average crack width to as low as one third of the previous value. Since in both arrangements the reinforcement was the same, the ratio of surrounding tensile concrete to total bar perimeter was also the same. Thus it has been demonstrated that a strictly formalistic application of equations for predicting crack widths which are based on the tension area to bar perimeter ratio may be quite misleading. Actually, in the beams having bunched reinforcement this ratio was infinitely large in the flange projections which were devoid of any longitudinal reinforcement.

Therefore, widely spaced cracks of great width probably originated there and then progressed toward the reinforced stem.

Diagonal tension cracking

The shear force at the appearance of the first diagonal tension crack may be predicted with reasonable reliability by the equation

$$V_{cr}' = \frac{I_g b}{Q_g} \times \frac{f_r f_c'}{f_r + 2f_c'}$$

for beams in which the M/Vd ratio is 3.0 or larger. It is apparent from this equation that to analyze shear behavior it is necessary to have information on the tensile strength of concrete (expressed here indirectly in terms of the modulus of rupture, f_r) as well as on the compression strength, f_c'.

With the exception of Beams IID-1 and IIIB-2, the width of the largest diagonal tension crack at service-load for all beams which appear in Table 7 was below 1/100 in. Both of the beams having larger diagonal tension cracks at the service-load had combination shear and flexural failures, emphasizing again that an ample reserve of shear strength over flexural strength is essential to good crack control. It is also worth noticing that in Beam IID-1M which had distributed negative tension reinforcement the width of the largest diagonal tension crack at the service-load is only one-sixth that of its companion specimen, Beam IID-1, which had bunched negative tension reinforcement. Thus with special attention to detailing of both the web and main reinforcement, it appears possible to keep diagonal tension crack widths at service-loads below prescribed limits.

No definite relationship between maximum stirrup stress and maximum diagonal tension crack width was found from the data obtained from the four beams whose stirrups were provided with strain gages. However, if the total external shear is assigned to the stirrups, it was found that a computed stirrup stress of 40,000 psi corresponded to a maximum crack width of about 1/100 in. in all four beams. It was these same four beams which had the distributed negative tension reinforcement.

ACKNOWLEDGMENTS

This investigation was carried out in the Department of Structural Engineering of the School of Civil Engineering of Cornell University. It was initiated and actively directed by Prof. George Winter, head of that department. The project was jointly sponsored by the United States Bureau of Public Roads and the Reinforced Concrete Research Council of the Engineering Foundation.

A task committee appointed by the Reinforced Concrete Research Council to provide general supervision of the investigation was constituted as follows: E. Hognestad, chairman; E. L. Erickson, W. H. Jacobs, R. C. Reese, I. M. Viest, and C. A. Willson, members.

The generous support of the investigation by the sponsors and the helpful suggestions of the task committee are gratefully acknowledged. Also, the writer wishes to express his appreciation of the valuable advice and encouragement of Dr. Winter, project director, and the assistance rendered by G. Schofield, H. Sujata, and Wei-Wen Yu, graduate assistants, and E. Pittman, laboratory mechanic.

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APPENDIX

Test specimens

The experimental program entailed the testing to destruction of 34 restrained T-beams and eight simply supported retest specimens. Thirty beams were acted upon by two concentrated loads and the remaining 12 by a series of closely spaced hydraulic rams to simulate the effect of a uniformly distributed load. The beams are shown in detail in Fig. 1a-1e and were divided into three series:

1. Series I beams were 17 ft long and consisted of a cantilever span of 4 ft and a span between supports of 10 ft. These beams were loaded at a point 3 ft from the interior reaction on the cantilever span and at a point 3 ft from the interior reaction on the span between supports.

2. Series II beams were 19 ft long and consisted of a cantilever span of 4 ft and a span between supports of 12 ft. These beams were loaded at a point 3 ft from the interior support on the cantilever span and at a point 6 ft from the interior reaction on the span between supports.

3. Series III beams were 20 ft long and consisted of a cantilever span of 5 ft 7 in. and a span between supports of 13 ft 5 in. These beams were loaded with a series of hydraulic rams spaced 1 ft on centers along the entire length of the beam (see Fig. A-1).

In the beams above, magnitudes and spacings of loads were so adjusted as to produce approximately equal maximum positive and negative moments. The beams of Series I were retested over their undamaged sections with two symmetrically placed, concentrated loads on a simple span of 8 ft. The loads were placed 2 ft apart and located 3 ft from the reactions. These portions of the Series I beams which were retested are designated with a suffix R.

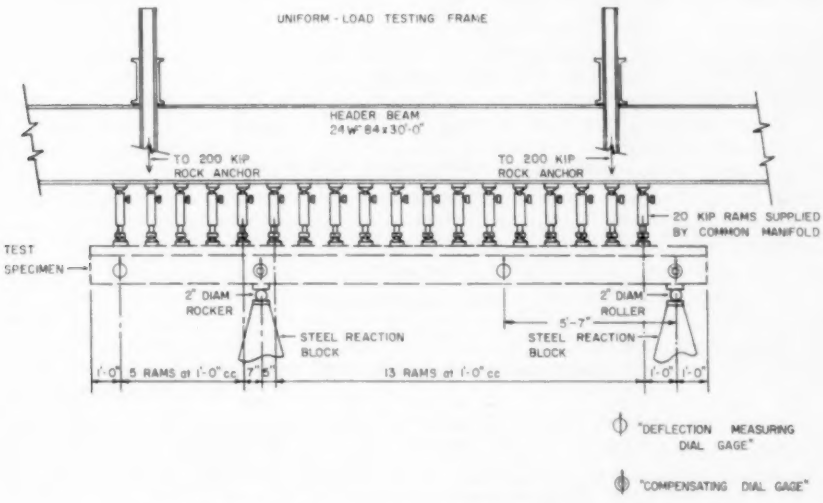

Fig. A-1—Loading arrangement for Series III beams

Properties of the beam specimens appear in Table 1. Fig. 1a-1e give the physical details for each series of beams. All specimens were of the same T-shaped cross-section with longitudinal tensile and compressive reinforcement consisting of alloy steel deformed bars having a nominal yield point of 80,000 psi. Six beams had plain webs and the remaining 36 had vertical stirrups for web reinforcement. Longitudinal steel percentages (referred to web of T-section) used in the beams were 1.37, 2.38, and 4.27 and web steel percentages were 0.25 and 1.63. Nominal concrete compressive strengths were 3000 and 5000 psi. Sections of maximum shear and maximum moment in the beams tested with two concentrated loads occurred over the interior support and under the interior concentrated load. A section of maximum moment and maximum shear occurred over the interior support in beams tested with uniformly distributed load system. A point of maximum moment also occurred between supports in these beams but the shear at such points was always zero.

Materials and fabrication

Two concrete strengths were used having nominal values of 3000 and 5000 psi. The mix proportions by weight for the 3000 and 5000 psi concretes were 1:4.35:4.60 and 1:2.43:2.43, respectively. The cement used was Type I portland cement. The cement content used was 5 bags per cu yd and w/c by weight was 0.79 for the 3000-psi concrete; for the 5000-psi concrete the cement content was 7 bags per cu yd and w/c by weight was 0.44. Coarse aggregate used was a mixture of natural gravel and crushed stone of 5/8 in. maximum size. All mixes were proportioned to produce their respective design strengths at 28 days and in such a way as to permit placement at a slump of 3 in.

Longitudinal reinforcement conformed to ASTM A 305 for deformations, was made of alloy steel, and supplied in two lots. The first lot, used to reinforce Series I, II, and III beams, was a silicon-manganese alloy steel (SAE

TABLE A-1—AVERAGE MECHANICAL PROPERTIES OF REINFORCING STEEL*

Bar size	Actual bar area, sq in.	Yield stress, psi	Ultimate stress, psi	Elongation, percent
Lot No. 1				
Web reinforcement				
2	0.073	76,700	85,700	9.9
4	0.184	49,400	86,370	21.2
Longitudinal reinforcement (SAE No. 9255)				
6	0.430	90,800	149,700	10.8
7	0.596	84,610	146,700	12.3
10	1.277	83,200	137,300	14.1
Lot No. 2				
Web reinforcement				
4	0.198	54,400	84,700	18.2
Longitudinal reinforcement (SAE No. 5150)				
5	0.310	103,100†	146,800	9.0
6	0.444	100,200†	148,700	17.4
7	0.630	84,200†	139,600	19.6

*Mechanical properties are averages for coupons taken from one-third of the total number of bars in each size group. Maximum standard deviation of sample data was in no case greater than 2 percent of the average value.
†Obtained by the 0.2 percent offset method.

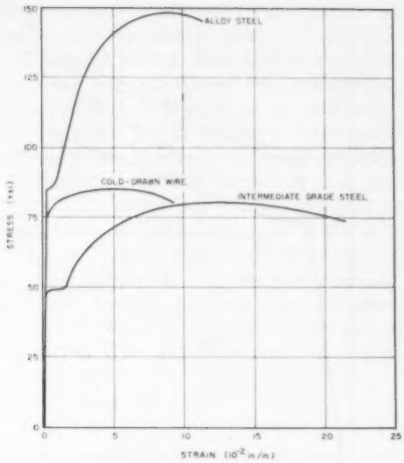

Fig. A-2 — Typical stress-strain curves for reinforcing bar material

No. 9255)* with a nominal yield stress of 80,000 psi. The curve designated "alloy steel" in Fig. A-2 is a typical stress-strain curve for the material of the first lot. The Series M beams were reinforced with #10 bars from the first lot and with #5, #6, and #7 bars from the second lot, which was a chromium alloy steel (SAE No. 5150)* also having a nominal yield stress of 80,000 psi. The stress-strain curves of the alloy steel bars from the second lot were similar to those of the first lot except that they were of the gradual-yielding type with no yield plateau. As such, they were similar in form to those for Tor-60 steel given by Wästlund.[15] Fig. A-3, which consists of initial portions of typical stress-strain curves for the material used to reinforce the Series M beams, shows the difference in yield behavior between the silicon-manganese alloy steel (#10 bar) and the chromium alloy steel (#5, #6, and #7 bars).

Two types of steel were used for web reinforcement: intermediate grade #4 bars with a nominal yield stress of 40,000 psi and cold-drawn wire #2 bars with a nominal yield stress of 80,000 psi. Typical stress-strain curves for this material are given in Fig. A-2. Average mechanical properties of all reinforcing bar material used in this investigation are given in Table A-1.

Beams were cast in the same positions as tested. Formwork consisted of braced sheet steel side members and 2-in. thick wood bases. Ends of forms were made of 3/4-in. plywood stock drilled to allow the longitudinal steel bars to extend 3 in. beyond the end face of the finished beam. The reinforcement was assembled outside of the forms into cages using soft-iron bar ties. After completion, the cages were lowered into the forms. Chairs were provided every 4 ft along the length of the beam to support the bottom bars. Actual beam dimensions were held to with 1/8-in. of design values in all cases. All wood surfaces of the forms were finished with several coats of waterproof varnish for sealing.

Materials were weighed out and concrete was mixed in the laboratory in a truck-mounted, horizontal, nontilting mixer. The concrete quantity supplied by one charge of the mixer was sufficient to cast from two to four beams at a

*See *Handbook of American Society for Metals*.[16]

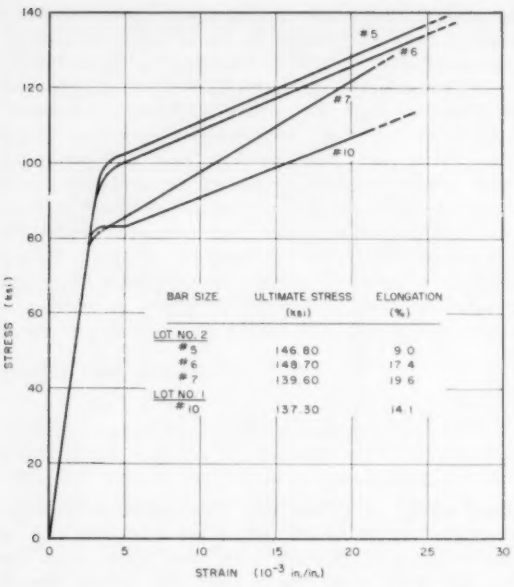

Fig. A-3—Initial portions of typical stress-strain curves for high-strength alloy steel bar material

time. Immediately before casting, all inside surfaces of the forms were given a light coating of oil. Forms were filled in two horizontal layers and the fresh concrete in each layer was thoroughly compacted with a 1/2-hp internal rod vibrator. Top surfaces of beams were screeded and smoothed with a steel bar. Two 6-in. coil loops were inserted in the top of each beam shortly after concreting to facilitate handling. A waterproof paper membrane was stretched over the top of each form 2 hr after casting, thus sealing in moisture on all sides of the beam. Three days after casting, forms were removed and the beams placed in the constant-humidity moist room for 18 days. Twenty-one days after casting, the specimens were removed from the moist room, stored in the laboratory, and allowed to dry until the day of the test. Average age of beam specimen at day of test was 28 days.

Nine 6 x 12-in. cylinders and four 6 x 6 x 21-in. modulus of rupture specimens were made from each batch of concrete. Two hr after casting, waterproof paper membranes were placed over the cylinder and the modulus of rupture forms exactly as in the case of the main beam specimens. The cylinder and modulus of rupture specimens were stripped, cured, and dried along with the corresponding main beam specimens. All cylinder and modulus of rupture specimens were tested at 28 days from time of casting.

Testing procedure

A standard 400,000-lb Baldwin universal testing machine was adapted to apply two concentrated loads to beam specimens by bolting a 20 ft steel reaction beam to the table and a 15 ft-9 in. load beam to the compression head of the machine. Two-in. diameter rockers or rollers were used at load and reaction points to transmit force to the beam. The uniform load condition was simulated by means of a series of concentrated loads spaced 1 ft on centers

along the entire length of the test specimen. These loads were produced by 10-ton hydraulic rams mounted in a special testing frame and fed from a common manifold which was in turn fed by a hand pump. Maximum operating pressure of the system was 10,000 psi. Ram load was measured by a Baldwin-Tate-Emery load cell and Bourdon gage apparatus attached to a "dummy" ram which was connected to the common manifold. Each ram was numbered and calibrated separately in place with a Morehouse 20,000-lb capacity proving ring. The difference between ram load measured with the proving ring and ram load indicated by the Bourdon gage was in all cases less than 1 percent of the measured load; consequently, no adjustment of indicated load data was deemed necessary in reporting test loads. Fig. A-1 gives details of spacing and method of application loads. Since rather large beam deflections were anticipated, a ball joint was provided at the top and bottom of each ram to allow it to incline slightly as the test specimen deflected. Maximum deviation of any one ram from its originally vertical position was approximately 2 deg at the end of the test.

Deflection measurements

Deflections were measured with dial gages reading to 1/1000 in. and having a maximum travel of 1.0 in. The dial gages were securely clamped to a steel bridge beam which was supported at the reaction points of the test specimen. The dial gage plungers impinged on the under side of the flange of the test specimen at a point close to the beam stem. Two sets of two dial gages each were provided. One set of dial gages, called "compensating dial gages" measured the displacement of the test specimen relative to the bridge beam due solely to vertical compression effects at the supports (i.e., displacement caused by seating of the test specimen in the immediate area of the reaction forces).

The other set of dial gages, called "deflection measuring dial gages," measured the displacement of the test specimen relative to the bridge beam due to deflection under load. These dial gages, of course, also registered displacements caused by the vertical compression effects. To remove the vertical compression effects from the deflection data, the readings of the deflection measuring dial gages were adjusted using the readings from the compensating dial gages. The location of the dial gages for Series III beams is shown in Fig. A-1. The deflection gage setup for the other beams was similar.

All Series I and Series II beams were tested in the 400,000-lb universal testing machine. After the beam was placed in the testing machine two steel plates were embedded in a plaster-of-Paris base on the top surface of the beam at the points of application of the load. While the plaster was still soft, the plates were accurately aligned and leveled. Testing commenced as soon as the plaster had hardened (approximately 3 hr after pouring). Loads were applied in equal increments to failure load. After each increment, the load was held at a constant value while deflection, strain, and bar end slip measurements were taken. At every other load increment widths of all cracks were measured in addition to the measurements cited above. At each load increment all cracks were outlined with a heavy black crayon and the machine load was recorded at the head of every crack. A photograph was taken of each beam showing the cracking pattern at failure. In all cases the beams were loaded until they ruptured completely. The testing time for each beam varied from 2 to 4 hr.

All Series III beams were tested in the uniform load apparatus described above. The testing procedure for these beams was substantially the same as that used in testing the concentrated load beams with the exception that the ram loading plates were not bedded in plaster-of-Paris.

Received by the Institute Apr. 18, 1960. Title No. 57-12 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 3, Sept. 1960 (Proceedings V. 57). Separate prints available at 75 cents each.

American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Dec. 1, 1960, for publication in the March 1961 JOURNAL.

Title No. 57-13

Effects of Aggregate Size on Properties of Concrete

By STANTON WALKER and DELMAR L. BLOEM

Research is described supplementing earlier work which indicated a lack of improvement in concrete strength resulting from increased maximum size of aggregate. The more recent program provided a broader range of classes of concrete and test variables for the purpose of checking the degree of applicability of the earlier finding.

The tests were made with maximum sizes of coarse aggregate ranging from 3/8 to 2 1/2 in., using three cement factors, from 4 to 8 sacks per cu yd, both with and without air entrainment. Compressive and flexural strength tests were made at ages of 7, 28, and 91 days, and specimens are available for test at 1 year. Tensile splitting tests were made at 28 and 91 days. Drying shrinkage measurements were also made on all concretes.

Results indicate that increasing the maximum size of coarse aggregate may not necessarily be beneficial to concrete strength. Drying shrinkage was not substantially increased by reduced size of aggregate down to about 3/4 in.

■ USE OF THE LARGEST PRACTICABLE SIZE OF AGGREGATE has long been considered a fundamental basis for economic concrete mixture design. The reason behind this long accepted concept is straightforward. Concrete made with larger sizes of aggregates requires less mixing water than with smaller sizes.[1,2,3] The well established and almost universally accepted "water-cement ratio law"[4] says that there should be an increase in strength consistent with the reduced water.

The assumption has been that the same water-ratio strength relationship applies "with given materials . . ., so long as the mix is of a workable plasticity"[4] without regard to grading. A critical review of information long available, inspired by recently developed data, now seems to make it clear that different levels of the relationship prevail for different gradings of the same aggregate. That such differences exist for different cements and different types of aggregate has long been accepted. The earlier evidence of the grading effect has been generally overlooked because of the broad and unqualified acceptance of the water-cement ratio law. The authors are indebted to Prof. H. J.

An Honorary Member of ACI, **Stanton Walker** is director of engineering for the National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D.C. During his career he has made many outstanding contributions in the fields of concrete and aggregate, among them the part he played in the development of the water-cement ratio and fineness modulus theories. He has been active in research, education, and development of specifications and test methods for concrete and aggregates. He was ACI president in 1947 and has served on various ACI administrative and technical committees, including the chairmanship of several.

Delmar L. Bloem is associate director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association. An active member of ACI, he is chairman of Committee 116, Nomenclature, and member of several other committees.

Gilkey for causing them to study the record. Much evidence supporting the "recent development," including data on which Abrams' *Bulletin* No. 1 on design of concrete mixtures[4] was based, has been unearthed.

No attempt will be made to summarize the early findings. The reader is referred to References 6-12. Briefly, the data cited afford convincing evidence that a higher level of the water-ratio strength relationship exists for well graded aggregates of smaller maximum sizes than for larger maximum sizes. Recent investigations[5,13-15] confirm these findings and go further. They demonstrate, so far as we know for the first time, that well graded aggregates may show higher strengths for smaller sizes than for larger ones when the cement factor and slump are the same. That is to say, the level of the water-cement ratio strength relationship for smaller sizes, within limitations, is sufficiently higher than for larger sizes that higher strengths are developed even for the same cement content and consistency.

SCOPE

This paper reports data expanding the scope of an earlier one presented before the Highway Research Board in 1959.[5] That paper dealt with non-air-entrained concrete, for the most part of only one cement content — 6 sacks per cu yd — tested at 28 days. Discussions following its presentation questioned whether or not the relationship, showing reduced strength with increased aggregate size, would be applicable alike to lower and higher cement contents, earlier and later ages at test, and air-entrained concrete. It was also asked if concrete made with the smaller sizes of aggregate might not show significantly higher drying shrinkage than with the larger sizes.

In an attempt to answer these questions the investigation reported here was made. It included: (1) both non-air-entrained and air-entrained concrete; (2) a wide range in cement factors; (3) ages at test up to 1 year (91-day tests are the latest now available); (4) flexural and compressive strengths; (5) drying shrinkage; and (6) as an interesting by-product, "splitting" tests of 6 x 12-in. cylinders, which provide a measure of tensile strength.

TABLE 1—CHARACTERISTICS OF AGGREGATES AS USED IN CONCRETE (SERIES 173)

Sieve size or item	Coarse aggregate,* maximum size				Fine aggregate†
	⅜ in.	¾ in.	1½ in.	2½ in.	
Grading, cumulative percents passing each size					
2½ in.	—	—	—	100	—
2 in.	—	—	—	85	—
1½ in.	—	—	100	70	—
1 in.	—	—	72	50	—
¾ in.	—	100	55	38	—
½ in.	—	63	35	24	—
⅜ in.	100	39	22	15	100.0
No. 4	0	0	0	0	97.4
No. 8	—	—	—	—	82.8
No. 16	—	—	—	—	69.8
No. 30	—	—	—	—	53.2
No. 50	—	—	—	—	23.2
No. 100	—	—	—	—	3.9
No. 200	—	—	—	—	1.1
Fineness modulus	6.00	6.61	7.23	7.77	2.70
Miscellaneous tests					
Unit weight, lb per cu ft	102.6	107.2	108.5	113.9	—
Voids, percent	37.5	34.7	33.9	30.6	—
Specific gravity:					
Bulk dry			2.63		2.63
Bulk SSD			2.64		2.64
Apparent			2.65		2.66
Absorption, percent			0.3		0.5

*Coarse aggregate separated into individual sizes and recombined to grading indicated for each batch.
†Sieve analysis represents average of ten tests.

As in the original studies, four maximum sizes of aggregate were employed: ⅜ in., ¾ in., 1½ in., and 2½ in. The grading of each was selected to conform with good practice and to meet conventional specification requirements. The coarse aggregate used was a hard, subangular, uniform quartz and quartzite gravel having an excellent service record. A siliceous sand of good quality, meeting conventional requirements for grading and physical properties, was used throughout. Characteristics of the aggregates are shown in Table 1.

The cement was a blend of equal amounts of five locally available brands. Because of limitations on cement storage facilities, the tests were made in two groups. Group 1, involving the non-air-entrained concretes, was made with one lot of this blend, and Group 2, the air-entrained concretes, with a second purchase of the same cements. In each case, the cements were blended by placing equal amounts of each brand in a 3½ S concrete mixer, sealing the mixer, and rotating it for a half hour or more. The blends thus secured were combined and kept in a tight bin until used.

With each of the four sizes of coarse aggregate, concrete was made with cement factors of 4, 6, and 8 sacks per cu yd in each of the two

TABLE 2—CHARACTERISTICS OF FRESH CONCRETE, SERIES 173*

Design cement, sacks per cu yd	Maximum size aggregate, in.	Calculated cement, sacks per cu yd	Mixing water, gal.		Aggregate proportioning			Slump, in.	Percent air	Weight, lb per cu ft
			per cu yd	per sack	F.M. of total aggregate†	b/b$_o$‡	Sand, percent total aggregate			
Group 1—Non-air-entrained concrete										
4	3/8	3.95	39.5	10.00	4.43	0.593	47.7	1.4	0.5	142.7
	3/4	3.97	36.0	9.07	5.00	0.661	41.2	2.2	1.8	145.9
	1 1/2	3.96	33.2	8.33	5.39	0.668	40.7	2.3	2.2	146.7
	2 1/2	3.97	32.5	8.19	5.68	0.634	41.3	2.1	2.2	146.9
6	3/8	5.95	37.5	6.30	4.50	0.598	45.5	2.1	2.3	145.3
	3/4	5.98	34.5	5.77	5.10	0.664	38.7	2.2	1.5	147.9
	1 1/2	5.94	33.2	5.59	5.50	0.668	38.3	2.8	1.5	148.8
	2 1/2	5.95	32.5	5.46	5.80	0.635	38.8	2.7	1.4	149.2
8	3/8	7.91	38.9	4.92	4.60	0.596	42.5	2.6	2.0	146.5
	3/4	7.86	36.1	4.59	5.22	0.656	35.5	2.7	1.9	148.5
	1 1/2	7.87	35.0	4.45	5.64	0.665	35.1	2.5	1.1	149.7
	2 1/2	7.84	34.5	4.40	5.96	0.628	35.7	2.7	1.4	149.7
Group 2—Air-entrained concrete										
4	3/8	3.89	37.7	9.69	4.42	0.585	47.8	1.8	4.5	140.6
	3/4	3.93	33.3	8.47	5.02	0.654	40.6	2.0	4.7	142.9
	1 1/2	3.93	31.1	7.91	5.41	0.661	40.1	2.1	4.7	143.7
	2 1/2	3.91	31.1	7.95	5.71	0.626	40.7	2.4	4.3	144.2
6	3/8	6.00	35.8	5.97	4.54	0.601	44.1	2.5	4.5	143.0
	3/4	6.00	32.2	5.37	5.14	0.666	37.7	2.1	4.8	144.1
	1 1/2	6.00	31.4	5.23	5.61	0.674	35.7	2.9	4.6	145.0
	2 1/2	6.01	30.5	5.07	5.93	0.640	36.3	2.7	4.6	145.1
8	3/8	8.03	37.2	4.63	4.69	0.604	39.8	3.2	4.7	142.9
	3/4	7.90	34.5	4.37	5.36	0.658	32.0	3.5	5.0	142.1
	1 1/2	8.03	34.4	4.28	5.83	0.676	30.8	3.1	4.2	145.2
	2 1/2	8.05	33.6	4.17	6.18	0.642	31.4	3.0	4.1	145.7

*Each value average for three batches mixed on different days.

†Fineness modulus of combined fine and coarse aggregate.

‡Ratio absolute volume of coarse aggregate per unit volume of concrete to absolute volume of coarse aggregate per unit volume of dry-rodded coarse aggregate (equivalent to dry-rodded volume of coarse aggregate per unit volume of concrete).

groups consisting of non-air-entrained and air-entrained concrete. The slump was maintained at approximately 2 to 3 in. and, for the air-entrained concretes, air content was controlled at 4.5 ± 0.5 percent by addition of an admixture at the time of mixing. Proportions of fine to coarse aggregates were selected on the basis of experience with the same materials in 6-sack non-air-entrained concretes in the several investigations of maximum size previously reported.[5] The quantities of coarse aggregate per unit volume of concrete thus determined were maintained the same for the other cement factors and for the air-entrained concretes with the same maximum size.

Three batches were made on different days for each of the 24 mixtures. All concrete was mixed in 3.5-cu ft batches for 6 min in a 3½ S tilting mixer. From each batch, six 6 x 12-in. cylinders, two 6 x 6 x 36-in. beams, and one 6 x 6 x 12-in. prism were molded. The cylinders were used for compressive strength tests at 7, 28, and 91 days and for tensile splitting tests[18] at 28 and 91 days. The beams, with each providing two tests, were used for flexural strength determinations at 7, 28, and 91 days. Drying shrinkage measurements were made on the 12-in. prisms. Specimens for 1 year compressive and flexural strength tests are still on hand.

Facilities limited to four the number of batches that could be mixed on any one day. Since the primary variable under study was aggregate size, a day's mixing was comprised of concrete made with each of the four coarse aggregate gradings of the same cement factor and air content class. About 2 months elapsed between mixing the two groups of tests. Mixing, curing, handling, and testing were carried out in strict accordance with applicable ASTM test methods.

DISCUSSION OF RESULTS

Average data on the tests of concrete are presented in Tables 2-5. Results for individual batches are available to those readers interested in making their own more detailed studies. Relationships among the variables are shown graphically in Fig. 1-9.

Table 2 summarizes the characteristics of the fresh concrete. The design criteria of cement factor, slump, and air content were closely maintained, assuring valid comparisons. It should be noted, however, that use of different lots of cement for the air-entrained and non-air-entrained concretes, even though of the same brands, may limit the comparability of strengths between the two classes of concrete. Average strength data through 91 days are given in Table 3. As mentioned previously, the 1-year results are not yet available. Table 4 presents the results of drying shrinkage and weight-change measurements. These specimens are undergoing continued observation. Table 5 gives data on density measurements of the fresh and hardened concrete.

Fig. 1 and 2 show the effects of maximum size of aggregate on compressive strength. Fig. 1 presents the data for the non-air-entrained concrete and Fig. 2 for the air-entrained. In Fig. 1 it will be seen that for 6- and 8-sack concretes the strengths for maximum sizes of 1½ and 2½ in. were always lower than for the ¾-in. size and generally lower than for ⅜ in. For the 4-sack concrete there was, with one exception, a general upward trend in strength with increased maximum size. The exception was the 91-day concrete made with the 2½-in. aggregate.

In Fig. 2 the same type of relationship is shown for the air-entrained concrete, with the smaller sizes appearing in an even more favorable light. These data again show a progressive reduction in strength with increased maximum size for the 6- and 8-sack mixes. Even in the 4-sack concrete, maximum sizes of ¾, 1½, and 2½ in. all gave substantially the same strength.

Fig. 3 and 4 present the flexural strength data in similar fashion. The pattern of the relationships is the same as for compressive strength, although the differences in strength, proportionately, are not so great.

Discussions by Lewis[14] and McNaughton[18] of the earlier paper,[5] suggested that the strength of the concrete might have an important effect

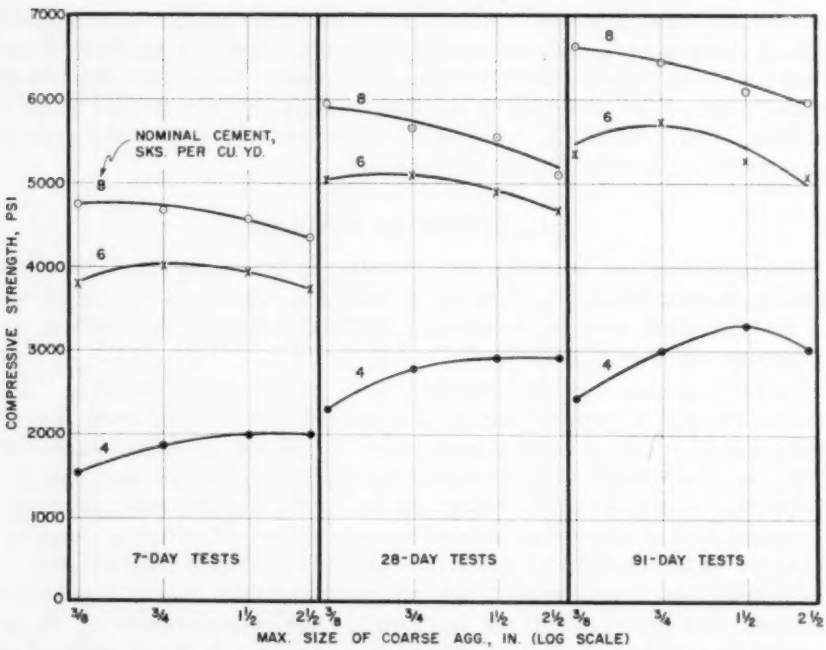

Fig. 1—Effect of size of coarse aggregate on compressive strength of non-air-entrained concrete (Series 173, Group 1)

TABLE 3—RESULTS OF STRENGTH TESTS, SERIES 173*

Design cement, sacks per cu yd	Maximum size aggregate, in.	Compressive strength of 6 x 12-in. cylinders, psi			Modulus of rupture of 6 x 6-in. beams, psi			tensile strength, psi (splitting)	
		7 days	28 days	91 days	7 days	28 days	91 days	28 days	91 days
Group 1—Non-air-entrained concrete									
4	⅜	1545	2320	2440	319	431	428	340	346
	¾	1875	2780	3000	351	491	483	389	409
	1½	1990	2910	3300	409	444	501	411	404
	2½	2010	2920	3015	362	494	498	426	408
6	⅜	3800	5035	5365	553	615	624	504	527
	¾	4025	5090	5740	598	658	633	528	517
	1½	3950	4915	5275	574	606	625	510	528
	2½	3725	4675	5195	523	624	617	517	526
8	⅜	4740	5950	6640	713	783	799	531	569
	¾	4680	5665	6455	692	771	802	485	547
	1½	4570	5545	6105	668	776	812	516	549
	2½	4350	5100	5970	662	718	773	515	544
Group 2—Air-entrained concrete									
4	⅜	1530	2365	2560	296	405	416	330	341
	¾	2125	3010	3300	370	472	495	403	408
	1½	2195	3110	3310	377	459	476	402	425
	2½	2145	2955	3140	357	464	483	365	403
6	⅜	3920	5115	5555	581	639	628	536	539
	¾	3715	4670	5235	573	624	611	562	490
	1½	3660	4680	5055	574	616	614	496	498
	2½	3710	4465	4555	566	568	608	494	505
8	⅜	4710	5700	6465	668	775	759	538	556
	¾	4110	4960	5595	649	673	710	469	471
	1½	3835	4700	5245	610	700	691	454	504
	2½	3845	4610	4980	630	664	684	442	497

*Each value average for tests of specimens from three batches mixed on different days.

TABLE 4—LENGTH AND WEIGHT CHANGES OF CONCRETE, SERIES 173*

Design cement, sacks per cu yd	Maximum size aggregate, in.	Non-air-entrained concrete				Air-entrained concrete	
		Shrinkage, percent		Weight loss, lb		Shrinkage in 6 weeks drying, percent	Weight loss in 6 weeks drying, lb
		6 weeks drying	14 weeks drying	6 weeks drying	14 weeks drying		
4	⅜	0.035	0.047	1.67	2.05	0.038	1.73
	¾	0.030	0.042	1.37	1.67	0.032	1.32
	1½	0.027	0.039	1.25	1.58	0.028	1.27
	2½	0.024	0.035	1.20	1.52	0.026	1.30
6	⅜	0.031	0.047	0.97	1.35	0.033	0.97
	¾	0.027	0.041	0.82	1.15	0.030	0.78
	1½	0.022	0.033	0.77	1.10	0.027	0.77
	2½	0.023	0.034	0.80	1.07	0.023	0.77
8	⅜	0.029	0.042	0.67	1.04	0.032	0.73
	¾	0.023	0.035	0.60	0.90	0.031	0.70
	1½	0.024	0.035	0.60	0.88	0.026	0.70
	2½	0.025	0.035	0.63	0.91	0.025	0.73

*Specimens 6 x 6 x 12-in. prisms moist cured 28 days before being exposed to air at 100 F; weights determined to nearest 0.05 lb; length changes measured with 0.0001-in. Whittemore strain gage to pairs of brass plugs on 10-in. centers in opposite faces of specimen. Each value average for three specimens from batches mixed on different days.

on the strength relationships, with the larger sizes showing to greater advantage at lower strength levels. Fig 1-4 suggest that this is so. At least the advantages for the smaller sizes became less marked as the quantity of cement was decreased. This is shown more clearly in Fig. 5. In that diagram, strengths are expressed as percentages of the strength of the comparable concrete made with the ⅜-in. size.

Fig. 5 shows clearly, however, that the relative advantages of the smaller sizes are not a function of strength alone. Observe Fig. 5B; the air-entrained concrete shows a greater spread than non-air-entrained between the smaller and larger aggregates even though the strength level was lower. Note in Fig. 5C that the advantage for the small aggregate was not changed by the higher strength resulting from an increase in age from 28 days to 91 days and was only mildly affected by the increase in age from 7 to 28 days. While the companion flexural strength data are not plotted, similar relationships would be shown.

That different water-ratio strength levels prevail for the different maximum sizes is shown clearly in Fig. 6. Without exception the level increased with reduction in maximum size. Further, on a percentage basis, the difference was as much for the higher water ratios as for the lower ones. Fig. 7 presents data which supplements both Fig. 5 and 6. Here both the compressive and flexural strengths are shown in relation to cement content and, for the first time, data of the splitting tests are presented graphically. For all three types of test the curve for the small size aggregate crosses those for the larger sizes, having

TABLE 5—DENSITIES OF FRESH AND HARDENED CONCRETE, SERIES 173*

Design cement, sacks per cu yd	Maximum size aggregate, in.	Density, lb per cu ft					
		Non-air-entrained concrete			Air-entrained concrete		
		Fresh	Hardened		Fresh	Hardened	
			Beams†	Cylinders		Beams†	Cylinders
4	⅜	142.7	145.8	145.5	140.6	143.7	143.9
	¾	145.9	148.7	148.3	142.9	146.2	145.8
	1½	146.7	150.0	149.1	143.7	147.0	146.6
	2½	146.9	149.6	149.6	144.2	147.1	147.4
6	⅜	145.3	148.3	148.2	143.0	146.0	145.6
	¾	147.9	150.6	150.5	144.1	146.4	146.6
	1½	148.8	151.5	151.2	145.0	147.9	148.1
	2½	149.2	152.3	151.0	145.1	147.9	147.4
8	⅜	146.5	149.0	149.1	142.9	145.6	146.0
	¾	148.5	151.2	151.0	142.1	145.1	145.3
	1½	149.7	151.8	151.6	145.2	147.5	147.4
	2½	149.7	152.2	151.7	145.7	148.3	147.5

*Fresh concrete tested for weight per cu ft in 0.5 cu ft measure weighed to nearest 0.05 lb, in accordance with ASTM C 138.

Hardened concrete specimens weighed in water and in air, saturated-surface-dry, at age of about 4 months—beams to nearest 0.05 lb and cylinders to 0.01 lb.

Each value average of three tests representing separate batches of concrete mixed on different days.

†6 x 6-in. x 23-28-in. pieces for 1-year flexural strength tests.

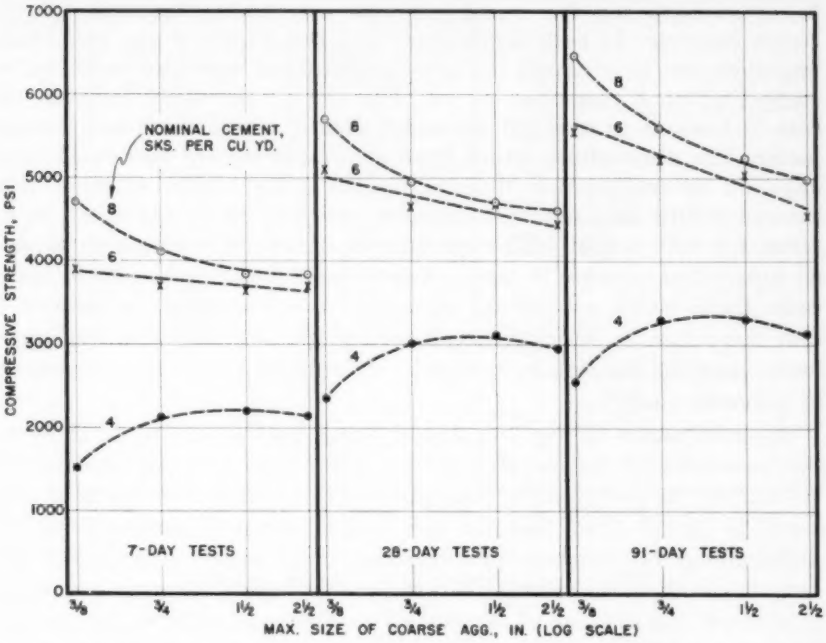

Fig. 2—Effect of size of coarse aggregate on compressive strength of air-entrained concrete (Series 173, Group 2)

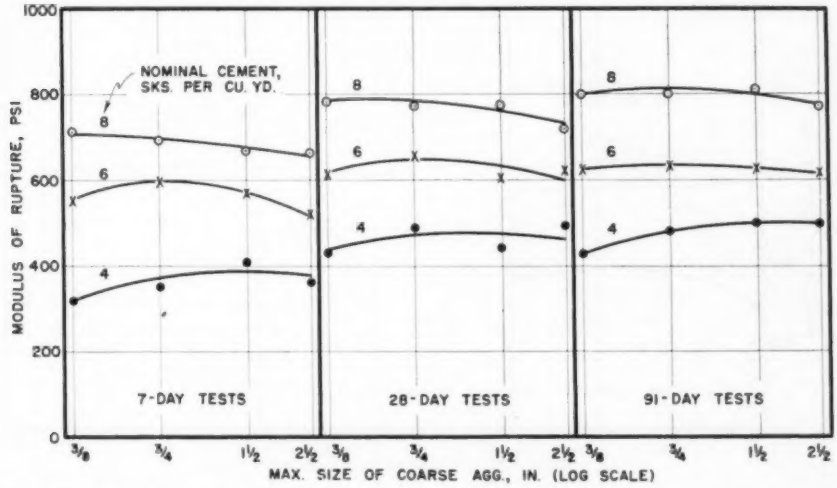

Fig. 3—Effect of size of coarse aggregate on flexural s ength of non-air-entrained concrete (Series 173, Group 1)

lower strength for the 4-sack concrete and higher strength for the 8-sack concrete. In both compression and flexure there was continuous improvement in strength of non-air-entrained concrete with added cement up to 8 sacks per cu yd. For the air-entrained concrete the rate of increase in strength decreased greatly above the 6-sack cement factor. The diminishing return from additional cement was much more apparent in compression than in flexure. The tensile splitting tests showed results somewhat inconsistent with the other measures. In all cases, for both non-air-entrained and air-entrained concrete there was no substantial increase in strength between 6 and 8 sacks and in many cases there was a substantial decrease. A further item of interest is that the range in strength displayed by the splitting test was much lower than for the flexure test, thus affording a less sensitive measure of concrete quality.

The information in Fig. 7 is supplemented by that in Fig. 8. Here the two measures of tensile strength, flexure, and splitting, are plotted in relation to compressive strength. Observe that the splitting test curve is flatter than that for flexural strength, re-emphasizing the shortcomings of that test as a measure of concrete quality. For the variables studied, the splitting test showed a range in strength, expressed in terms of the median, of 55 percent as compared with 65 percent for flexure and 90 percent for compression.

The drying shrinkage data available at this time are presented in Fig. 9. They represent measurements on 6 x 6 x 12-in. prisms cured

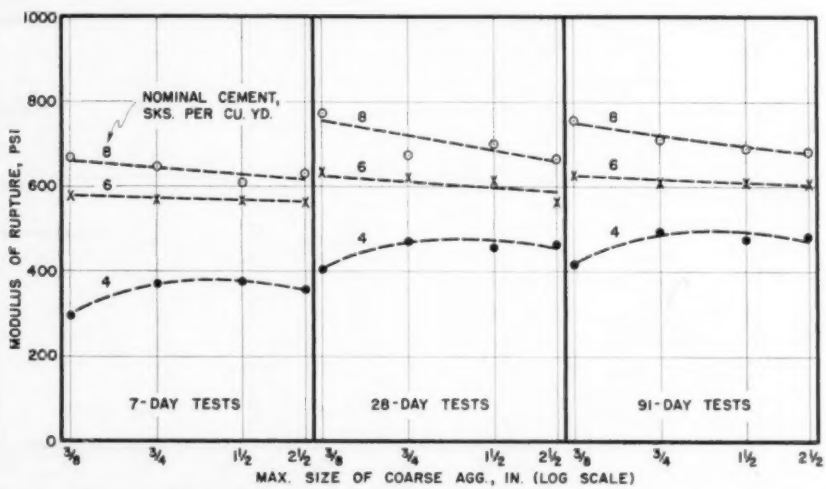

Fig. 4—Effect of size of coarse aggregate on flexural strength of air-entrained concrete (Series 173, Group 2)

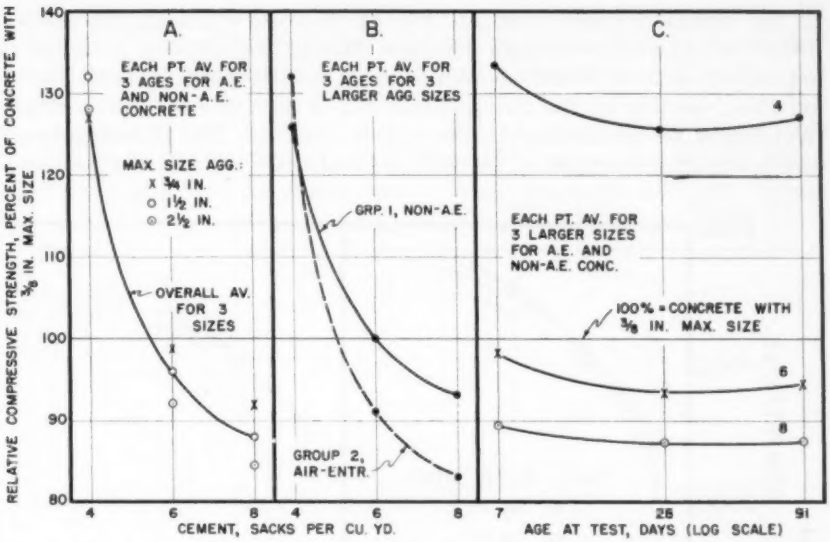
A.
B.
C.
EACH PT. AV. FOR 3 AGES FOR A.E. AND NON-A.E. CONCRETE
MAX. SIZE AGG.:
X 3/4 IN.
○ 1½ IN.
◎ 2½ IN.
OVERALL AV. FOR 3 SIZES
EACH PT. AV. FOR 3 AGES FOR 3 LARGER AGG. SIZES
GRP. 1, NON-A.E.
GROUP 2, AIR-ENTR.
EACH PT. AV. FOR 3 LARGER SIZES FOR A.E. AND NON-A.E. CONC.
100% = CONCRETE WITH 3/8 IN. MAX. SIZE
RELATIVE COMPRESSIVE STRENGTH, PERCENT OF CONCRETE WITH 3/8 IN. MAX. SIZE
CEMENT, SACKS PER CU. YD.
AGE AT TEST, DAYS (LOG SCALE)

Fig. 5—Relative concrete strength levels in relation to concretes with 3/8 in. maximum size coarse aggregate (Series 173)

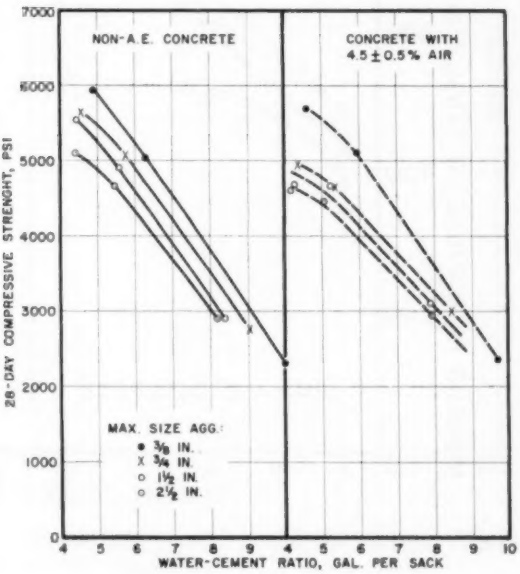

Fig. 6 — Water ratio versus strength for different maximum sizes of aggregate (Series 173)

in the standard moist room at 73 F for 28 days and then air dried at 100 F for 6 weeks. Length changes during the drying period were measured with a 0.0001-in. Whittemore strain gage to brass inserts on 10-in. centers in the two vertical 6 x 12-in. faces of the specimens. Weights were determined to the nearest 0.05 lb. The shrinkage measurements are expressed as percentages and the weight losses in pounds.

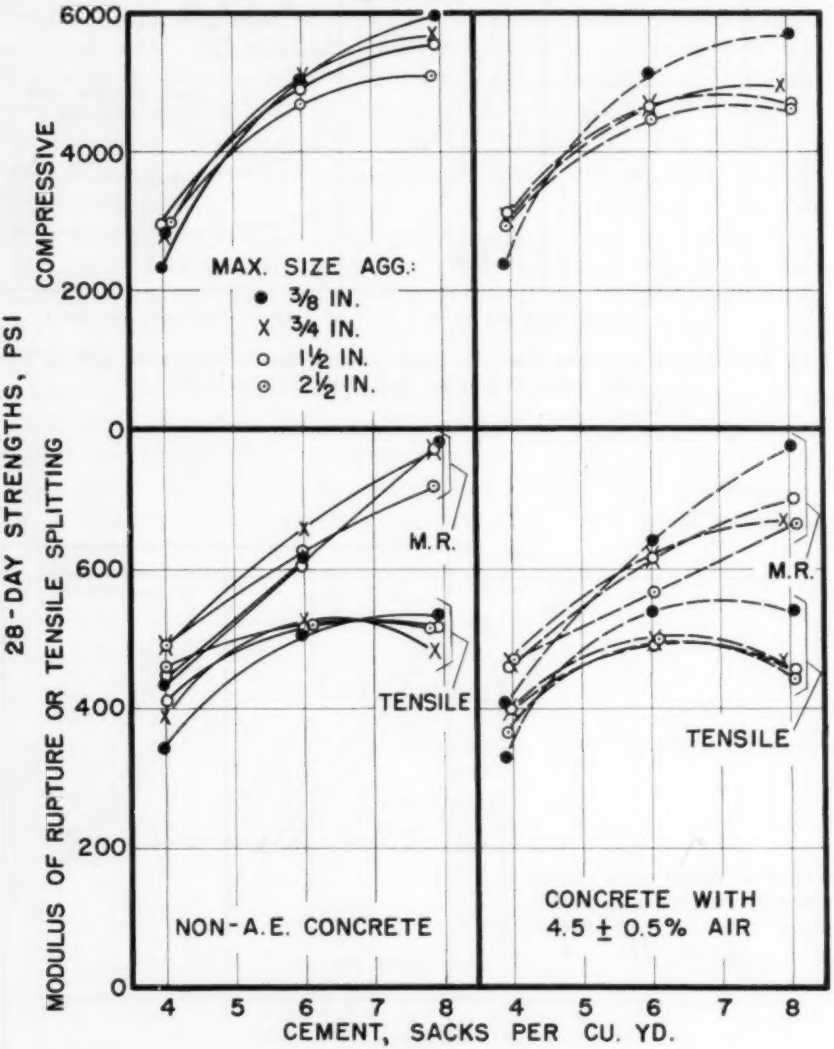

Fig. 7—Effect of added cement on strength for various maximum sizes of aggregate (Series 173)

A 1 lb change in weight corresponds approximately to 2½ percent by weight or 6½ percent by volume of the concrete. Fig. 9 shows that shrinkage and loss of water during drying increased with decreasing cement factor and with decreasing aggregate size. These relationships may be modified after longer drying. In the range of aggregate sizes from ¾ in. to 2½ in., differences in shrinkage amounted to only about 0.005 percentage points, which would seem to be of little or no sig-

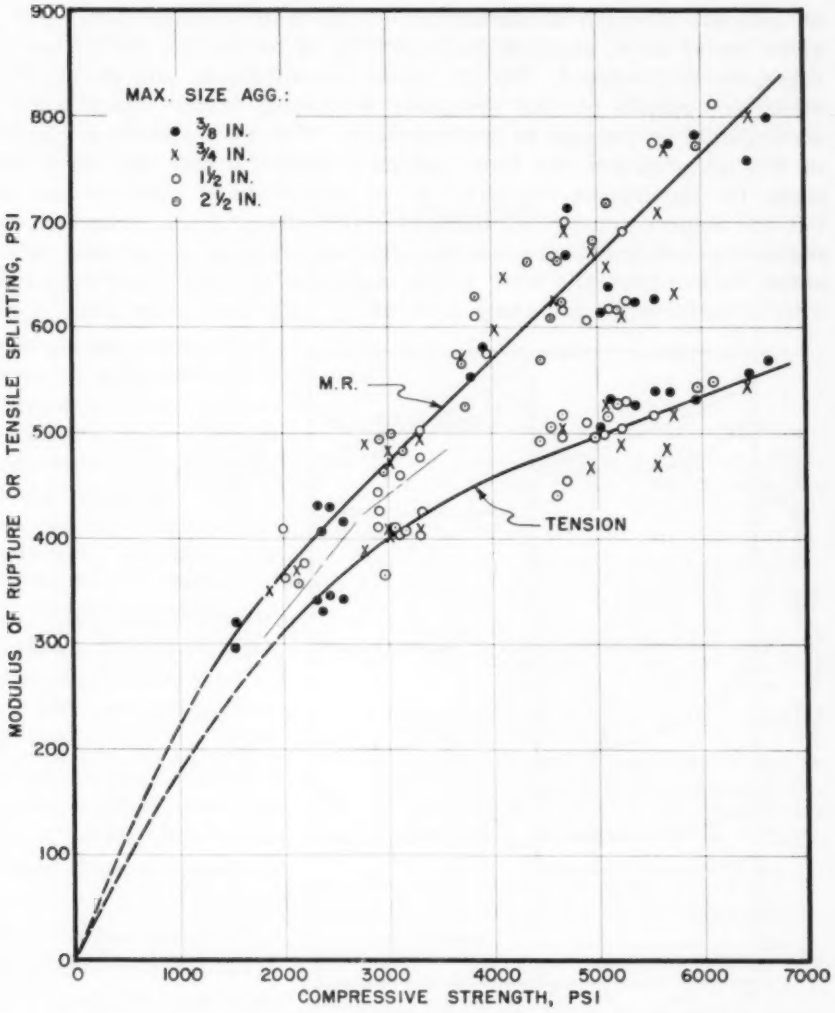

Fig. 8—Relationships of compressive to flexural and tensile strengths of concrete (Series 173)

nificance in substantially all concrete constructions. In fact, the differences due to change in aggregate size do not appear to be significantly greater than those resulting from substantial changes in cement factor. Air entrainment apparently had little effect on either the drying shrinkage or loss of water.

SUMMARY AND CONCLUSION

The tests clearly show that size of coarse aggregate exerts an influence on concrete strength independently of the water-cement ratio. For a given water ratio, strength becomes less as maximum size of coarse aggregate is increased. For the same cement factor and slump, this effect is opposed by the reduction in mixing water which always accompanies an increase in aggregate size. With the materials employed in this investigation, the latter influence predominated only in a few cases, for the leanest mixtures. In all air-entrained concrete and all but the leanest non-air-entrained concrete, strength was progressively reduced as maximum size was increased above ¾ in. or, in most cases, above ⅜ in. Even the lean, 4-sack concretes in most cases exhibited little or no strength advantage from use of aggregate larger than ¾ in.

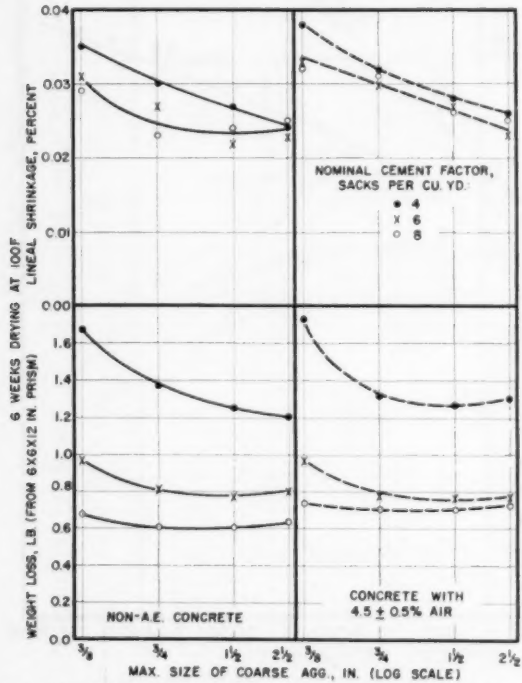

Fig. 9—Effect of size of coarse aggregate on drying shrinkage of concrete (Series 173)

It seems probable that the relationship between strength and maximum size will vary depending upon the characteristics of the aggregate. However, work in our laboratory and data reported by other investigators, involving a variety of aggregates differing widely in mineral and physical makeup, show that a point is likely to be reached at a moderate size level beyond which there is no strength advantage in using larger sizes.

The decreased strength resulting from the use of large sizes was most pronounced in the concretes of higher cement factors.

However, that strength level alone was not the controlling factor is indicated by comparisons between air-entrained and non-air-entrained concretes and between tests at different ages. The air-entrained concretes show relatively greater strength reductions for the larger sizes in spite of their lower strength level. Tests at 7, 28, and 91 days revealed a relatively small effect of age on the size relationship in spite of substantial differences in strength with age.

The preliminary drying shrinkage data indicate increased volume changes resulting from use of small size aggregate. As expected, the drying shrinkage was closely related to quantity of mixing water and hence reflected the greater amounts required by the small-size materials. For the range of maximum sizes from ¾ in. to 2½ in., differences in drying shrinkage were probably so small as to be of little practical significance in most concrete.

ACKNOWLEDGMENT

The tests were performed in the research laboratory sponsored jointly by the National Sand and Gravel Association and the National Ready Mixed Concrete Association at the University of Maryland. They were conducted under the immediate direction of R. D. Gaynor, associate research engineer and laboratory manager, and Herman L. Knoppel, Jr., assistant laboratory manager, and under the general direction of the authors.

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Received by the Institute Mar. 2, 1960. Presented at ACI 56th annual convention, New York, N.Y., Mar. 16, 1960. Title No. 57-13 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 3, Sept. 1960 (Proceedings V. 57). Separate prints are available at 60 cents each.

American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Dec. 1, 1960, for publication in March 1961 JOURNAL.

Title No. 57-14

Transfer of Bending Moment Between Flat Plate Floor and Column

By JOSEPH DI STASIO, SR. and M. P. VAN BUREN

A method is presented for calculating the maximum unit shearing stresses, measuring both diagonal tension and punching shear, due to combined gravity load and bending in flat plate floor slabs about exterior and interior columns. Provision is made for the effect of openings in the column head region. The values of allowable stresses are discussed, and recommendations made for a suggested test program.

■ THE PROBLEM OF THE DETERMINATION of and safe values for the stresses due to a joint moment at the junction between a flat plate floor slab and a column is of importance especially in view of the increasing tendency toward the use of small columns. We made an extensive investigation of this question for one of our projects in 1949 and developed the following method of analysis which we believe will provide a satisfactory solution.

Notations

A = effective area of peripheral section [see Eq. (4f) and (4g)]
A_c = concrete area of peripheral section
a_1 and a_2 = distances from centroid of A to extreme fibers
B = width of spandrel beam
b = width of critical section for diagonal tension
(b) = perimeter of peripheral section
c = side of critical section for diagonal tension
d = effective depth of slab
D = effective depth of spandrel beam
e = distance of centroid of peripheral section from column center
g = distance of centroid of section from midpoint of c
H = horizontal shear in column
h = story height
j = ratio of distance between centroid of compression to centroid of tension to depth d
J = polar moment of inertia of peripheral section about its centroid
J_c = polar moment of inertia of concrete in peripheral section
M, M_1, and M_2 = resisting moments on width of section
m = residual balanced floor moments on width of section, which do not effect joint rotation, but effect the resisting moment capacity of slab

The late **Joseph Di Stasio, Sr.** as a partner in the New York consulting firm of Di Stasio and Van Buren, effectively promoted and widened the field of reinforced concrete construction through development of design methods and devotion of effort in professional societies and code committees. For his outstanding work he received several awards, including the Concrete Reinforcing Steel Institute Award in 1955 for his developmental work in flat plates. Active in ACI committee work he was awarded posthumously the Henry L. Kennedy Award in 1959.

ACI member **M. P. van Buren** has been a partner in Di Stasio and van Buren since 1924. He has coauthored various papers on flat slab design. He is a member of ACI Committee 318, Standard Building Code, and also a member of the Reinforced Concrete Research Council's Task and Project Advisory Committee on Investigations of Multiple Panel Reinforced Concrete Floor Slabs.

n = ratio of modulus of elasticity of steel to that of concrete

O_1, O_2 = excluded lengths of peripheral section due to openings

p = ratio of area of reinforcement to area of concrete

S = total depth of spandrel beam

t = total slab thickness

V = total shear around peripheral section

v, v_1, and v_2 = unit shearing stresses

X = distance of opening from critical section, and arm of torsion couple $Vx = T$

y = distance from centroid of column to face b

z = centroid of peripheral sections above bottom of slab

Primed letters

b', c', etc. refer to peripheral section at column face

Σw = total width of openings parallel to edge of critical section

x' = $y'd_e$ = distance of section under consideration from face of column

d_e, in = (t − 1½), in.

EXTERIOR COLUMN

Fig. 1a shows the plan of a panel with particular reference to an exterior column. Two sections through the slab require consideration. The first, which follows a periphery parallel to the column faces at a distance t — 1½ in. therefrom, is the critical section for diagonal tension. The second, taken at the column perimeter along the faces c', b', c' is the section for maximum punching shear. It is required to determine the stresses and their distribution on both of these sections.

Fig. 1b shows a section through the floor at the exterior column in equilibrium under the moments and forces as indicated. The total moment at the center of the joint is given by Hh, and includes the effect of any eccentric loads. At the critical section for diagonal tension, this moment is resisted by a moment M on face b, a shearing couple Ve in which V is the total shear around the peripheral section and e is the distance of the centroid of the section from the column centroid, and a torsion T on the peripheral section.

The equilibrium of moments is expressed by the following equation:

$$Hh = M + Ve + T \quad \text{.....(1)}$$

Similarly for the section at the column face, the equation is

$$Hh = M' + V'e' + T' \quad \text{.....(1a)}$$

Considering first Eq. (1), the limiting value of M may be taken as the resisting moment of the section of width b, which equals $K_c bd^2$ or $K_s bd^2$ depending on whether concrete or steel controls, and may include the effect of compression steel if present.* Its value may be increased by the addition of more steel if required. That it exceeds the proportion of the normal slab moment assumed in the design to be acting on this width need cause no concern as it is a local condition produced by the concentration of stresses at the column. With this moment determined, Eq. (1) may be rewritten as

$$T = Hh - M - Ve \quad \ldots (2)$$

For the determination of the stresses, V and T, which both involve shear, should be considered together. For a case of this kind, in which the contact surface of the slab against the column is prevented from warping, and in which the lengths of peripheral sections increase rapidly with the distance out from the column face, the St. Venant type torsion formula is inapplicable. It is believed that a method using the polar moment of inertia of the peripheral section offers the simplest and most satisfactory solution.

For the determination of the stress distribution, the force V and the moment T may be represented by two equal and opposite forces V at the centroid of the peripheral section and a couple $Vx = T$ as

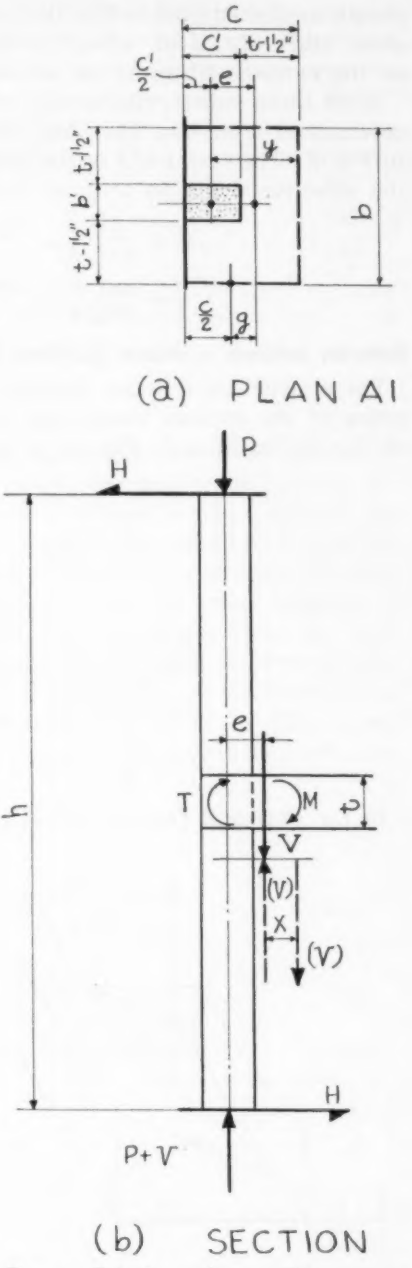


Fig. 1—Exterior column with no spandrel beam

*While this assumption may not accord with elastic theory for early stages of loading, ultimate load considerations indicate that stresses determined on this basis will provide a measure of the safe capacity of the section.

shown by dashed lines in Fig. 1b. V/A then gives the average unit vertical shear and $-Ta_1/J$ or $+Ta_2/J$ give the additional unit vertical shears on the extreme fibers of the section of depth t.

With these values, the maximum unit shears, using $j = 7/8$, may be determined from Eq. (3a) and (3b), and are indicated isometrically in Fig. 2. The factor t/d is required to transform A and J in terms of the effective depth to conform with concrete design practice.

$$v_1 = \frac{8t}{7d}\left[\frac{V}{A} - \frac{(Hh - M - Ve)\,a_1}{J}\right] \quad \text{(3a)}$$

$$v_2 = \frac{8t}{7d}\left[\frac{V}{A} + \frac{(Hh - M - Ve)\,a_2}{J}\right] \quad \text{(3b)}$$

Exterior column without spandrel beams

For a structure without spandrel beams, the evaluation of the properties of the section, consisting of three planes cut through the slab on the dotted lines in Fig. 1a, is as follows:

$$e = y - \frac{c}{2} + g \quad \text{(4)}$$

$$g = \frac{bc}{2\,(2c + b)} \quad \text{(4a)}$$

$$a_1 = \frac{c}{2} + g \quad \text{(4b)}$$

$$a_2 = \frac{c}{2} - g \quad \text{(4c)}$$

$$A_c = (2c + b)\,t \quad \text{(4d)}$$

$$J_c = \frac{2tc^3}{12} + \frac{2ct^3}{12} + 2ctg^2 + bt\left[\frac{c}{2} - g\right]^2 \quad \text{(4e)}$$

In Eq. (4d) and (4e) A_c and J_c represent the area and polar moment

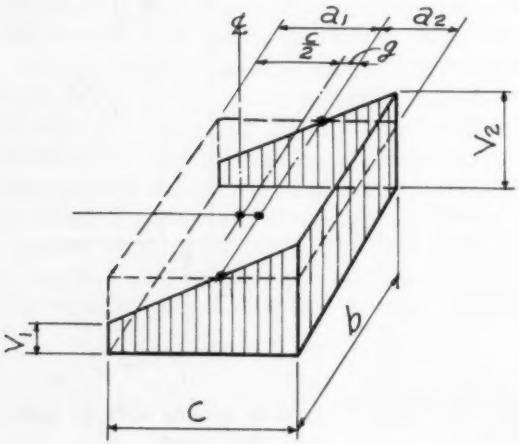


Fig. 2—Shear stress diagram of column with no spandrel beam

of inertia of the concrete alone. When unit shears have the low values permissible in flat plate construction, considerable additional resistance can be developed through the dowel action of reinforcing bars crossing the section, and it may be reasonable to increase the values of A_c and J_c by the factor $1 + (n - 1)\,p$ in which p is the percentage of total (top and bottom) slab steel crossing the perimeter of the peripheral section. On this basis

$$A = A_c[1 + (n - 1)\,p] \quad \text{(4f)}$$

$$J = J_c[1 + (n - 1)p] \quad \text{(4g)}$$

Where there is much variation in the distribution of the steel crossing the various faces of the section, proper values of the factor $1 + (n - 1)\,p$ may be applied to the individual terms of Eq. (4d) and (4e) rather than to the equations as a whole.

Exterior column with spandrel beams

For structures with spandrel beams, conditions are more favorable. First, spandrel beams are usually located near the outer face of the column with an eccentricity counter to the floor moment so that the moment Hh in Fig. 1b is reduced. Second, the additional concrete section of the beam increases the area A_c. Third, this additional concrete section and its more effective location increases the polar moment of inertia J_c. Fourth, when the spandrel beam is deeper than the slab, the effective width of the critical section for diagonal tension parallel to the beam may be increased by twice the projection of the beam above or below the slab. The revised conditions are shown in Fig. 3, and the properties of section are given by Eq. (4) and in Eq. (5) and (5a) to (5e).

$$z = \frac{(2c + b)\,\frac{t^2}{2} - 2B\,(S - t)\left[\frac{S}{2} - \frac{t}{2}\right]}{(2c + b)\,t + 2B\,(S - t)} \quad \text{(5)}$$

$$g = \frac{bt\,\frac{c}{2} + 2B\,(S - t)\left[\frac{c}{2} - \frac{B}{2}\right]}{(2c + b)\,t + 2B\,(S - t)} \quad \text{(5a)}$$

$$a_1 = \frac{c}{2} + g \quad \text{(5b)}$$

$$a_2 = \frac{c}{2} - g \quad \text{(5c)}$$

$$A_c = (2c + b)\,t + 2B\,(S - t) \quad \text{(5d)}$$

$$J_c = \frac{2ct^3}{12} + \frac{2ct^3}{12} + \frac{2B\,(S - t)^3}{12} + \frac{2(S - t)\,B^3}{12} + 2ctg^2$$
$$+ 2ct\left[\frac{t}{2} - z\right]^2 + bt\left[\frac{c}{2} - g\right]^2 + 2B\,(S - t)\left[\frac{c}{2} - \frac{B}{2} + g\right]^2$$
$$+ 2B\,(S - t)\left[\frac{S}{2} - \frac{t}{2} + z\right]^2 \quad \text{(5e)}$$

Eq. (4f) and (4g) may apply as before except that if there is much difference between the values of p in spandrel beam and slab, proper values of the factor $1 + (n - 1)\,p$ should be applied to the individual terms of Eq. (5d) and (5e) rather than to the equations as a whole to allow for the distribution of the steel.

Eq. (3a) and (3b) also apply as before except that in Eq. (3a), the factor t/d should be replaced with S/D to provide for the greater depth of spandrel beam.

For the section at the column face, all of the above formulas apply with substitution of proper values for M and the dimensions to conform with the smaller periphery.

INTERIOR COLUMN

Fig. 4a shows the plan at an interior column. Here again two sections require consideration — one at distance $t - 1\frac{1}{2}$ in. out from the column faces for diagonal tension, and one following the perimeter of the column faces for punching shear.

Fig. 4b is a section through the floor in equilibrium under the action of the moments and forces shown. This figure resembles Fig. 1b except that clockwise resisting moments M_1 and M_2 occur on opposite sides of the column along faces b. In addition, the balanced floor moment, equal to the smaller of the gravity load column strip moments on the two faces of the column, remains in the slab. A fraction of this, in proportion of width b to column strip, acts on each of the faces b and is designated m. At the left face, m is of opposite sense to M_1, and must first be overcome before any new stresses are introduced in the structure. Further resistance at this point is a function of the bottom slab steel which should be anchored into the column to obtain full development of this moment. At the right face, m is of same sense as M_2 and must be deducted from it to determine the available resistance. Although m is added to M_1 and deducted from M_2 and cancels out in the equations of equilibrium, it must be considered in determining the available resisting moments on each face. The equation of equilibrium then becomes

$$T = Hh - (M_1 + m) - (M_2 - m) \qquad (7)$$

The maximum unit shears are given by

$$v_1 \text{ or } v_2 = \frac{8t}{7d}\left[\frac{V}{A} \pm \frac{Hh - (M_1 + m) - (M_2 - m)}{J}\,\frac{c}{2}\right] \qquad (8)$$

and are shown isometrically in Fig. 5.

The properties of the section are

$$A_c = 2(c + b)\,t \qquad (9a)$$

$$J_c = \frac{2tc^3}{12} + \frac{2ct^3}{12} + 2bt\left[\frac{c}{2}\right]^2 \qquad (9b)$$

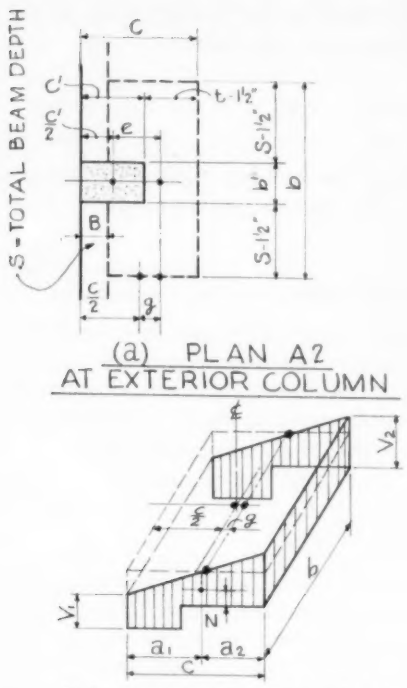


Fig. 3 (above)—Exterior column with spandrel beam

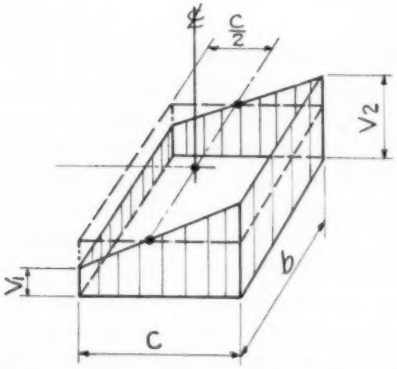


Fig. 5 — Shear stress diagram for an interior column

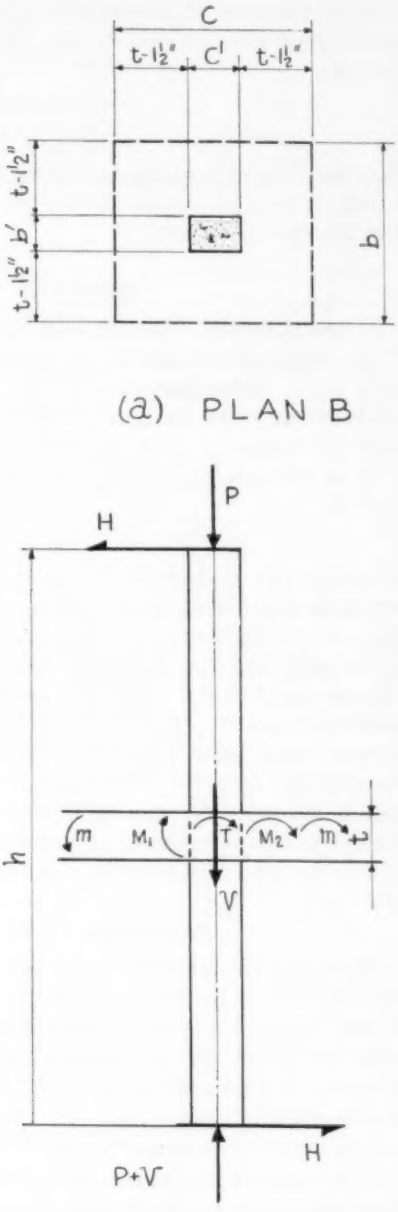


Fig. 4—An interior column

As in the case of the exterior column, to allow for dowel action of the reinforcing bars, A_c and J_c may be increased by the factor $1 + (n - 1)p$, so that

$$A = A_c [1 + (n - 1) p] \quad (9c)$$

$$J = J_c [1 + (n - 1) p] \quad (9d)$$

Similar formulas apply to the section at the column face, with substitution of proper values of M_1, M_2, m, and dimensions to conform with the smaller periphery.

GENERAL CONDITIONS

In the formulas for both exterior and interior columns, the condition of an external moment in one direction only has been considered acting at a joint. When moment in two directions at right angles occur simultaneously, the analysis for each direction may be made independently and the resulting values added to give the final maximum shear.

If no moment is present, Eq. (3a), (3b), and (8) reduce to

$$v = \frac{V}{(b)\ jd\ [1 + (n - 1)\ p]}$$

in which (b) represents the perimeter of section, and v is the intensity of vertical shearing stress thereon assumed uniformly distributed. The factor $1 + (n - 1)\ p$ in this expression, and as a modifier for A_c and J_c, is proposed for use with the low shear values permissible at the column head in flat slabs. It is a direct function of the amount of steel provided and is supplementary to the variation in allowable shearing stresses now permitted from rearrangement or grouping of the bars. In addition to the vertical shear, v, there may also be balanced horizontal shearing stresses present on the peripheral section due to the torsional action inherent in the slab itself but which do not materially affect the magnitude of the critical vertical shear.

OPENINGS THROUGH FLOOR SLABS

Openings through the floor slab in the column head region should generally be avoided. However, where unavoidable, provision for them in the computation of stresses can be made by proper modification of g, A, and J in the formulas to allow for the part of section omitted. In this connection, if the opening is unsymmetrically placed with respect to the column center line, Eq. (7) and (8) should be further modified by the addition or deduction of Ve from Hh as in Eq. (2), (3a), and (3b).

The location of openings also has an important bearing on the strength of the construction. Two cases require consideration, and are illustrated in Fig. 6. First, any opening or series of openings of total width Σw at a distance x (less than $\Sigma w/2$) outside an edge of the critical section, should be considered to reduce the effective length

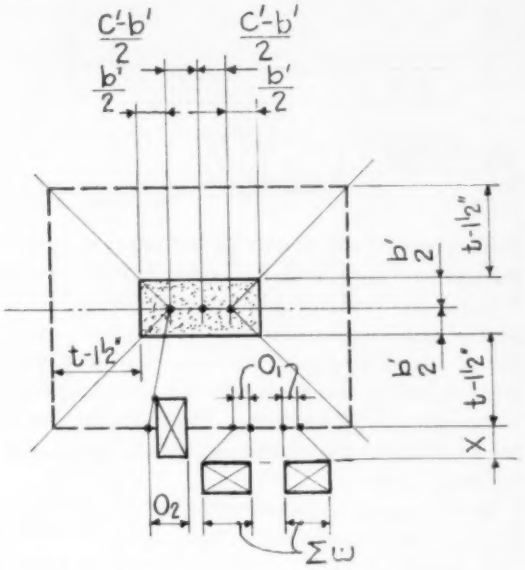


Fig. 6—Openings in slab adjacent to column

of that edge by an amount $O_1 = \Sigma w - 2x$. Second, for openings extending inside the critical section, any point on the periphery at $t - 1\frac{1}{2}$ in. from the column face, which cannot be connected by a straight line not crossing the opening with a point on the long axis of the column within a distance $(c' - b')/2$ from the column centroid, should be excluded from the effective section offering resistance to diagonal tension. In this expression, c' is the long side and b' the short side of the column, and O_2 in the figure represents the excluded length. The lengths O_1 and O_2 must be deducted from the perimeter in the computations of g, A, and J for the section. Where openings are present, stresses should be investigated both for punching shears at the column face, and for diagonal tension at the critical section.

In determining shearing stresses inside the critical section, O_1 and O_2 may be considered as the effective openings distant $t - 1\frac{1}{2}$ in. from the column face, and all points which do not fall on straight lines not crossing these openings as described for O_2 should be excluded from any resisting section closer than $t - 1\frac{1}{2}$ in. to the column face.

PERMISSIBLE STRESSES

The shearing strength of the construction increases rapidly between the peripheral section distant $t - 1\frac{1}{2}$ in. from the column and the face of the support. As the face is approached, more and more lines of diagonal compression abut solidly against it to carry the load and relieve diagonal tension stresses as shown in Fig. 7. Curves illustrating

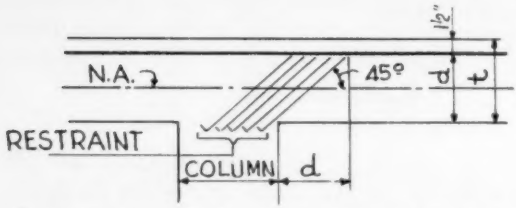


Fig. 7 — Lines of diagonal compression

the variation of shearing stresses in this region due to loading are shown in Fig. 8 and 9. These curves are based on peripheral sections formed by vertical planes equidistant from and parallel to the faces of the supporting column. It is to be noted that the formulas of this paper directly apply only to complete peripheral sections so located, and on which the resistance to shearing deformation may be considered to be essentially uniform. They should not be used without modification for sections in which the planes are unequally spaced from the respective column faces.

With respect to permissible stresses on the critical section for diagonal tension, the present ACI regulations for flat slabs prescribes a maximum unit shear of $0.03f_c'$ when at least 50 percent of the required column strip steel crosses the section. This value must be reduced proportionately when less steel crosses the section to $0.025f_c'$ when the minimum of 25 percent of the column strip steel is so disposed. With torsion, somewhat higher shear values might be acceptable because: (a) torsion contributes no additional direct tensile stresses as occur in beams below the neutral axis, and (b) the maximum stresses occur only on the extreme fibers of the section. However, until further information is available, it will be wise to limit shears to the maximum values at present specified.

At the column face, which is subjected to punching shear with no diagonal tension, much higher shearing values can be permitted. Punching shear tests are difficult to make because specimens usually fail in direct stress before the ultimate strength in pure shear is reached, and few reliable results have been reported. The 1920 ACI Code recommended values for punching shear of $0.10\ f_c'$ on the bd section but reduced to $0.06f_c'$ when the depth of the supported member is less than 1/15 of the span. Shearing strength undoubtedly varies with the amount of direct compression acting on the same plane which provides a friction force to increase the resistance. As the result of a recent investigation, values of $0.079f_c'$ when $f_c = 0$, and $0.156f_c'$ when $f_c = 0.5\ f_c'$ have been suggested as lower limits for the shearing strength of concrete. However, far higher values are attained in everyday practice, and are tacitly permitted by codes. For example, where a direct compressive stress of $0.45f_c'$ is permitted on the extreme fiber of a beam, this is accompanied by stresses on planes at 45 deg to the extreme fiber of $v = f_c = 0.225\ f_c'$

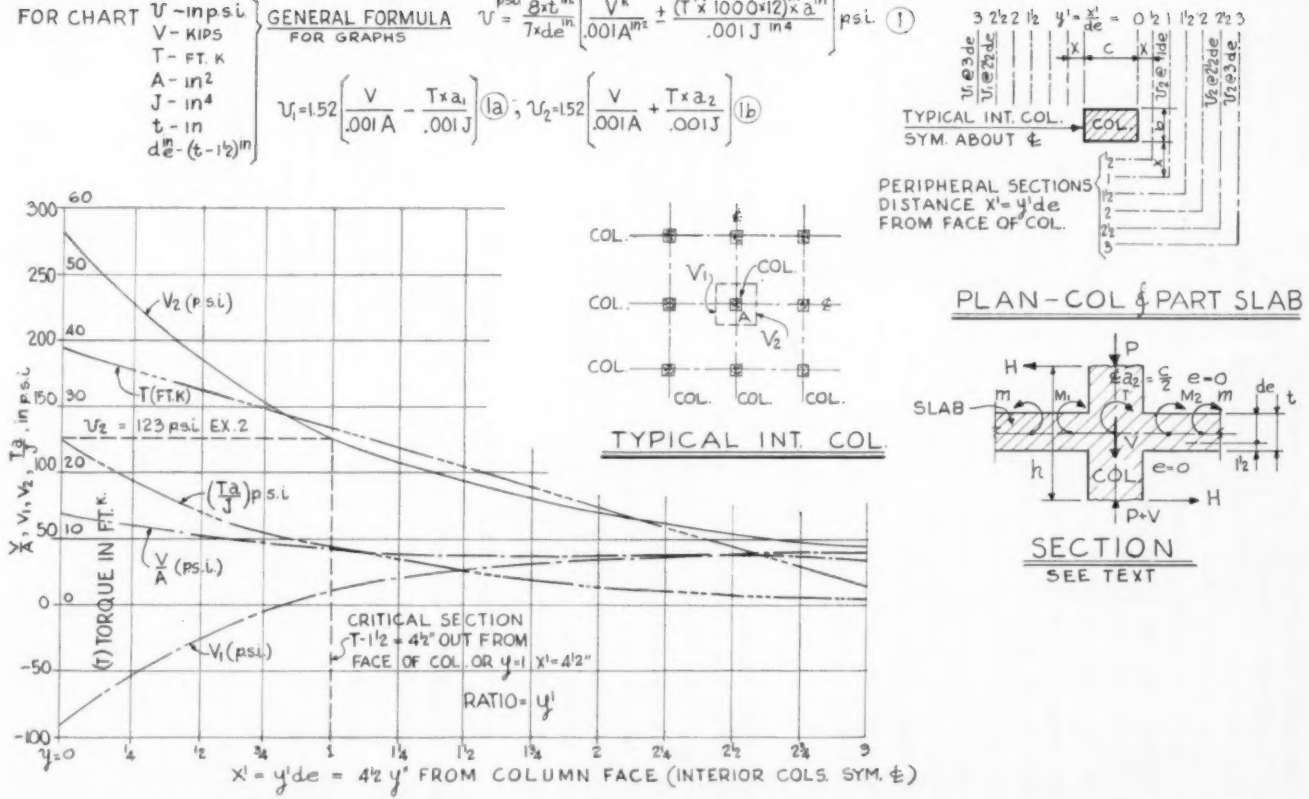

FOR CHART v - in p.s.i.
V - KIPS
T - FT. K
A - in²
J - in⁴
t - in
d_e^m - (t - 1½)^in
GENERAL FORMULA FOR GRAPHS
v = 8 x t / (7 x d_e) [V/(.001 A) ± (T x 1000 x 12) x a / (.001 J)] p.s.i. (1)
v1 = 1.52 [V/(.001 A) − T x a1/(.001 J)] (1a) ; v2 = 1.52 [V/(.001 A) + T x a2/(.001 J)] (1b)
TYPICAL INT. COL. SYM. ABOUT ℄
PERIPHERAL SECTIONS DISTANCE X' = y' de FROM FACE OF COL.
PLAN - COL & PART SLAB
SLAB
SECTION
SEE TEXT
TYPICAL INT. COL.
V2 (p.s.i.)
T(FT.K)
V2 = 123 p.s.i. EX. 2
(Ta/J) p.s.i.
V/A (p.s.i.)
V1 (p.s.i.)
(T) TORQUE IN FT. K.
CRITICAL SECTION T-1½ = 4½" OUT FROM FACE OF COL. OR y=1 X'=4½"
RATIO = y'
X' = y' de = 4½ y' FROM COLUMN FACE (INTERIOR COLS. SYM. ℄)

Fig. 8—Typical interior column (symmetrical about the center line) showing maximum and minimum shearing or diagonal tension intensities at various peripheral sections from column face, and other characteristics

These are safe stresses, but how much they may be exceeded is unknown. If a flexural member is assumed cracked to the neutral axis, the vertical shear may be considered distributed parabolicly over kd. Taking $k = 0.4$, and extreme fiber stress $f_c = 0.45f_c'$, the maximum unit shear $v_{max} = (3/2)(V/0.4bd)$, which occurs at the midpoint of kd where $f_c = 0.225f_c'$. By the usual equation, $v = (V)/(7/8bd) = 8/7 \times 2/3 \times 0.4\, v_{max} = 1/3.3\, v_{max}$. If $v_{max} = f_c = 0.225\, f_c'$ is accepted as a safe criterion, the safe value of v is $0.225f_c'/3.3 = 0.068f_c'$.

In the examples accompanying this paper, $0.0625f_c'$, which is ¼ the allowable stress in direct compression, has been used for comparative purposes as a reasonable minimum safe value for punching shear stress. With this value it will usually be found that, even with the smaller section involved, punching shear is not controlling, and that the critical stresses occur at the section for diagonal tension.

When wind or earthquake forces are involved, permissible shearing stresses may be increased 33 percent as now permitted by codes.

RECOMMENDATIONS

It is recommended that a series of tests be undertaken to establish safe values for the stresses occurring in the transfer of moment between column and slab. These tests should cover slabs and columns of various practical sizes at both interior and exterior supports, with and without spandrel beams, openings, and with varying amounts of reinforcement to check dowel action. Pending the results of such tests, it is believed that the method and formulas described herein will lead to safe and satisfactory designs.

APPENDIX — TYPICAL EXAMPLES

Example 1—Exterior column

Given an exterior column in a flat plate panel as shown in Fig. 1, in which $f_c' = 3500$ psi, $f_s = 20{,}000$ psi, $b' = 14$ in., $c' = 24$ in., $t = 6$ in., slab shear = 16 kips, total wall load shears = 9 kips located 8 in. outside column center, combined wind and gravity floor moment at joint center = 57.8 ft-kips. Determine unit vertical shears on extreme fibers of critical section for diagonal tension when six #6 top bars and four #5 bottom bars cross each of the three faces of the section.

From Fig. 1 and Eq. (4) and (4a) to (4g), the properties of the section are

$$b = 14 + 2 \times 4.5 = 23 \text{ in.}; \quad c = 24 + 4.5 = 28.5 \text{ in.};$$

$$g = \frac{23 \times 28.5}{2\,(2 \times 28.5 + 23)} = 4 \text{ in.}$$

$$e = (28.5 - 12) - \frac{28.5}{2} + 4.1 = 6.35 \text{ in.}$$

$$a_1 = \frac{28.5}{2} + 4.1 = 18.35 \text{ in.}; \quad a_2 = \frac{28.5}{2} - 4.1 = 10.15 \text{ in.}$$

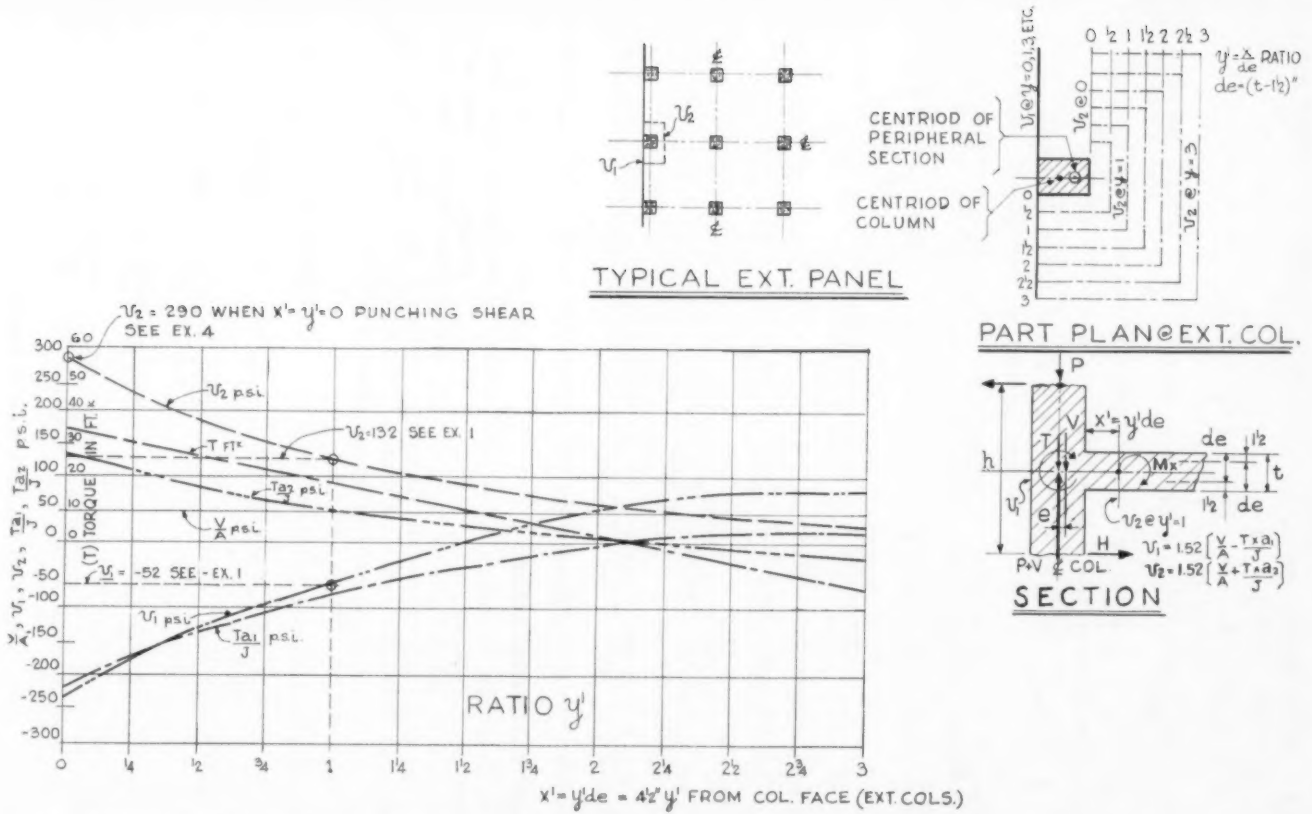


Fig. 9—Typical exterior column showing maximum and minimum shearing or diagonal tension, stress intensities at various peripheral sections from column face, and other characteristics

$$A_c = (2 \times 28.5 + 23)\,6 = 480 \text{ sq in.}; \quad J_c = \frac{2 \times 6 \times 28.5^3}{12} + \frac{2 \times 28.5 \times 6^3}{12}$$

$$+ 2 \times 28.5 \times 6 \times 4.1^2 + 23 \times 6 \left(\frac{28.5}{2} - 4.1\right)^2 = 44{,}141 \text{ in.}^4$$

$$p = \frac{3\,(6 \times 0.44 + 4 \times 0.31)}{480} = 0.024; \quad A = 480\,(1 + 7.5 \times 0.024) = 566 \text{ sq in.}$$

$$J = 44{,}141\,(1 + 7.5 \times 0.024) = 52{,}086 \text{ in.}^4$$

The resisting moment on face b is the smaller of the following values:

$$M = f_s jdA_s = 20 \times \tfrac{7}{8} \times \frac{4.5}{12} \times 6 \times 0.44 \times 1.33 = 23 \text{ ft-kips}$$

or

$$M = \left[Kbd^2 + f_c \frac{kd - d'}{kd} (2n - 1)\, A_s' \frac{d - d'}{12} \right] \times 1.33 = \left[0.273 \times 23 \times 4.5^2 \right.$$

$$\left. \times \tfrac{1}{12} + 1.575 \times \tfrac{1}{2} \times 16 \times 4 \times 0.31 \times 3.5 \times \tfrac{1}{12} \right] \times 1.33 = 20.1 \text{ ft-kips}$$

$$Hh = 57.8 - 9 \times \frac{8}{12} = 51.8 \text{ ft-kips}$$

The torsion is given by

$$T = Hh - M - Ve = 51.8 - 20.1 - 25 \times 6.35 \times \tfrac{1}{12} = 18.5 \text{ ft-kips}$$

From Eq. (3a) and (3b)

$$v_1 = \frac{8 \times 6}{7 \times 4.5} \left[\frac{25{,}000}{566} - \frac{18{,}500 \times 12 \times 18.35}{52{,}086} \right] = 1.52 \left[44 - 78 \right] = -52 \text{ psi}$$

$$v_2 = \frac{8 \times 6}{7 \times 4.5} \left[\frac{25{,}000}{566} + \frac{18{,}500 \times 12 \times 10.15}{52{,}086} \right] = 1.52 \left[44 + 43 \right] = 132 \text{ psi}$$

Permissible stress for combined wind and gravity load = $100 \times 1.33 = 133$ psi.

Example 2—Interior column

Given an interior column in a flat plate panel as shown in Fig. 4, in which $f_c' = 3500$ psi, $f_s = 20{,}000$ psi, $b' = 14$ in., $t = 6$ in., $V = 33$ kips, combined wind and unbalanced gravity moment = 57 ft-kips, and balanced column strip moment = 3.6 ft-kip per ft of width. Determine maximum unit vertical shear on extreme fiber of critical section for diagonal tension when six #6 top bars and four #5 bottom bars cross each of the four faces of the section.

From Fig. 4 and Eq. (9a) to (9d)

$$b = 14 + 2 \times 4.5 = 23; \quad c = 24 + 2 \times 4.5 = 33; \quad A_c = 2(23 + 33)\,6 = 672 \text{ sq in.}$$

$$J_c = \frac{2 \times 6 \times 33^3}{12} + \frac{2 \times 33 \times 6^3}{12} + 2 \times 23 \times 6 \left[\frac{33}{2} \right]^2 112{,}266 \text{ in.}^4$$

$$p = \frac{4(6 \times 0.44 + 4 \times 0.31)}{672} = 0.023; \quad A = 672(1 + 7.5 \times 0.023) = 786 \text{ sq in.}$$

$$J = 112{,}266\,(1 + 7.5 \times 0.023) = 132{,}000 \text{ in.}^4$$

The moments on faces b are

$$M_1 = f_s jdA_s = 20 \times \tfrac{7}{8} \times \frac{4.5}{12} \times 4 \times 0.31 \times 1.33 = 10.8 \text{ ft-kips}$$

$$M_2 = 20.1 \text{ ft-kips (see Example 1)}$$

The torsion is given by

$$T = Hh - (M_1 + m) - (M_2 - m) = 57 - \left(10.8 + 3.6 \times \frac{23}{12}\right)$$
$$- \left(20.1 - 3.6 \times \frac{23}{12}\right) = 26.1 \text{ ft-kips}$$

From Eq. (8)

$$v = \frac{8 \times 6}{7 \times 4.5}\left[\frac{33{,}000}{786} + \frac{26{,}100 \times 12 \times 16.5}{132{,}000}\right] = 1.52\left[42 + 39\right] = 123 \text{ psi}$$

Permissible stress for combined wind and gravity load = 100 × 1.33 = 133 psi.

Example 3—Opening at interior column

Determine maximum unit vertical shear on extreme fiber of critical section for diagonal tension in Example 2 when two 6 in. wide openings, 10 in. apart, are symmetrically placed on one face *c* and extend 2 in. inside the critical section.

$$\text{Each distance } O_2 = 6 \times \frac{11.5}{9.5} = 7.3 \text{ in.}$$

$$A_c = 672 - 2 \times 7.3 \times 6 = 584 \text{ sq in.}$$

$$J_c = 112{,}266 - 2 \times 7.3 \times 6 \times 8.5^2 = 105{,}916 \text{ in.}^4$$

$$p = 0.023 \times \frac{672}{584} = 0.0265; \quad A = 584\,(1 + 7.5 \times 0.0265) = 700 \text{ sq in.}$$

$$J = 0.105916\,(1 + 7.5 \times 0.0265) = 127{,}500 \text{ in.}^4$$

$$v = \frac{8 \times 6}{7 \times 4.5}\left[\frac{33{,}000}{700} + \frac{26{,}100 \times 12 \times 16.5}{127{,}500}\right] = 1.52\,(47 + 41) = 133 \text{ psi}$$

Permissible stress for combined wind and gravity load is 100 × 1.33 = 133 psi.

Example 4—Punching shear at exterior column

As a corollary to Example 1, determine maximum punching shear at the column face for the same loading. With $b' = 14$ in., $c' = 24$ in., the formulas give

$$g = \frac{14 \times 24}{2(48 + 14)} = 2.72 \text{ in.}; \quad e = 12 - 12 + 2.72 = 2.72 \text{ in.}$$

$$a_1 = 12 + 2.72 = 14.72 \text{ in.}; \quad a_2 = 12 - 2.72 = 9.28 \text{ in.}$$

$$A_c = (2 \times 24 + 14) \times 6 = 372 \text{ sq in.}$$

$$J_c = \frac{2 \times 6 \times 24^3}{12} + \frac{2 \times 24 \times 6^3}{12} + 2 \times 24 \times 6 \times 2.72^2$$
$$+ 14 \times 6 \times 9.28^2 = 24{,}045 \text{ in.}^4$$

$$A = 372 \times 1.18 = 440 \text{ sq in.}; \quad J = 24{,}045 \times 1.18 = 28{,}400 \text{ in.}^4$$

$$M = 20.1 \times {}^{14}\!/_{23} = 12.2 \text{ ft-kips}; \quad Hh = 57.8 - 9 \times {}^{8}\!/_{12} = 51.8 \text{ ft-kips}$$

$$T = 51.8 - 12.2 - 25 \times 2.72 \times {}^{1}\!/_{12} = 33.9 \text{ ft-kips}$$

$$v = \frac{8 \times 6}{7 \times 4.5}\left[\frac{25{,}000}{440} + \frac{33{,}900 \times 12 \times 9.28}{28{,}400}\right] = 1.52\left[57 + 133\right] = 290 \text{ psi}$$

Permissible stress = 0.0625 × 3500 × 1.33 = 291 psi.

TABLE A-1—SHEAR AND MOMENT BETWEEN COLUMN AND SLAB, WITH AND WITHOUT OPENINGS

Section	V capacity, kips	M capacity, ft-kips
Solid section, diagonal tension	25	57.8
Section with holes, diagonal tension	14.3	45.7
Solid section, punching shear	25.0	57.8
Section with holes, punching shear	19.0	37.2

Example 5—Opening at exterior column

Determine the shear and moment capacity at the joint in Example 1 when 24 in. long openings cover each face c' of the column.

Applying the rule for openings extending inside the critical section as described for O_2 in Fig. 6, the peripheral section of Fig. 1a is restricted to face b for diagonal tension and b' for punching shear.

Considering the openings as deductions from the section, Eq. (4) and (4a) to (4g) at the critical section for diagonal tension reduce to $b = 23$ in., $c = g = a_1 = a_2 = J = 0$, $e = 28.5 - 12 = 16.5$ in.; and

$$A = bt\,[1 + (n - 1)\,p] = 23 \times 6\,(1 + 7.5 \times 0.024) = 163 \text{ sq in.}$$

As the section has no width in the direction of the applied moment, the resistance is limited to shear and moment on face b. Eq. (3a) and (3b) reduce to $v = 8t/7d \times V/A$. With the allowable $v = 133$ psi for combined wind and gravity loads as before,

$$V = 133 \times \frac{7}{8} \times \frac{4\frac{1}{2}}{6} \times 163 = 14{,}300 \text{ lb}$$

From Eq. (1), $Hh = M + Ve = 20.1 + 14.3 \times 16.5 \times 1/12 = 39.7$ ft-kips. Adding the moment due to eccentricity of $9 \times 8/12 = 6$ ft-kips gives 45.7 ft-kips as the combined wind and floor moment at the joint center.

At the column face, $b = 14$ in., $e = 12$, $A_c = 14 \times 6 = 84$ in.^{4}, $A = 84\,(1 + 7.5 \times 0.024) = 99$ sq in., $M = 20.1 \times 14/23 = 12.2$ ft-kips, and the allowable $v = 0.0625 \times 3500 \times 1.33 = 291$ psi. From which,

$$V = 291 \times \frac{7}{8} \times \frac{4\frac{1}{2}}{6} \times 99 = 19{,}000 \text{ lb}$$

$$Hh = M + Ve = 12.2 + 19.0 \times \frac{12}{12} = 31.2 \text{ ft-kips}$$

Adding 6 ft-kips for eccentricity, the combined wind and floor moment at the joint center is 37.2 ft-kips.

A comparison of the results with those for the solid section can be seen in Table A-1. It is apparent that the section with holes is governed by shear and has a capacity of 14.3/25 or 57 percent of the solid section.

Received by the Institute Sept. 9, 1958. Title No. 57-14 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 3, Sept. 1960 (Proceedings V. 57). Separate prints are available at 60 cents each.

American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Dec. 1, 1960, for publication in the March 1961 JOURNAL.

Title No. 57-15

Resistance to Shear of Reinforced Concrete Beams

Part 2 — Beams with Vertical Stirrups

By J. TAUB and A. M. NEVILLE

The redistribution of internal forces following the yield of stirrups in a simply supported reinforced concrete beam is described, and it is shown that after the stirrups have yielded the shear strength of a beam depends on the resistance of the compression zone above the diagonal tension crack and the tension zone at the lower end of the crack. Thus the shear strength of a beam is not proportional to the amount of web reinforcement; this is discussed in considerable detail.

The second role of the stirrups in the shear resistance of a beam is their ability to resist the splitting of the beam along the tension steel; for this an effective restraint of the tension steel by the stirrups is essential.

Different types of shear failure are described as well as the influence of various factors on the behavior of reinforced concrete beams failing in shear. Tests at the University of Manchester show the higher strength in shear of T- and L-beams compared with rectangular beams. The influence of cutoff of the tension steel on the shear strength of simply supported beams is described. Suggestions are made on the provision of vertical stirrups in simply supported reinforced concrete beams.

■ VERTICAL STIRRUPS HAVE BEEN COMMONLY USED as shear reinforcement for a long time, and there is available a considerable amount of experimental data on the strength of beams with a variety of stirrup types, sizes, and spacings. The interpretation of the amassed evidence is, however, not always clear; it is proposed now to consider the distribution of forces within a beam with vertical stirrups at various stages of loading.

The generally accepted function of stirrups is to resist the opening or widening of the diagonal tension cracks, and thus to prevent the failure of a reinforced concrete beam due to shear. It may be relevant to observe in this connection that as long as 40 years ago it was shown that beams differing only in web reinforcement exhibit first cracking at the same load.[2] * It is only after this cracking has started that the behavior of beams depends on the type of web reinforcement, and the ultimate strength in shear is, *inter alia*, a function of this reinforcement.

*References are numbered consecutively beginning with Part 1 of this five-part series.

Notation

α = angle of inclination of bent-up bars to axis of beam
A_s = cross-sectional area of tension reinforcement
A_w = cross-sectional area of web reinforcement in one plane
a = shear span
b = width of rectangular beam, or web width of T-beam
C = resultant (inclined) compression force in concrete
C' = horizontal component of the compression force in concrete
d = effective depth of beam
f_c' = compressive strength of concrete test cylinder
f_{cu} = compressive strength of concrete test cube
jd = lever arm of internal forces in beam
kd = depth of compression zone of concrete
P_u = total load on beam at failure
P_s = total load on beam at first slip of the end of tension steel
$p = \frac{A_s}{bd} \times 100$
$r = \frac{A_w}{bs \sin\alpha} \times 100$
S = resultant stirrup force
s = spacing of web reinforcement measured parallel with axis of beam
T = force in main tension reinforcement
T_h = horizontal component of T
T_v = vertical component of T
V = total shearing force
V_c = shearing force resisted by concrete above neutral axis
v = nominal shearing stress in concrete
v_u = nominal shearing stress at failure

REDISTRIBUTION OF INTERNAL FORCES

The mode of formation of diagonal tension cracks was discussed in detail in Part 1,† and the deformation of the main tension steel, where intersected by the crack, in beams *without* web reinforcement is well known.

Let us now consider a beam with vertical stirrups. At low loads strain measurements have shown tension in the upper portions of the stirrups and compression in the lower portions, due respectively to transverse contraction and expansion induced by flexure.[27] As soon as the diagonal tension crack has formed, these strains become wholly tensile and continue to increase in magnitude with increase in the applied load. Fig. 15 shows the deformation of a beam with vertical stirrups after the diagonal tension crack has extended to the level of tension steel. From the nature of the formation of the crack it is apparent that the deformation of the stirrup, where it crosses the diagonal tension crack, has to be in the direction of the opening, and there is thus a shearing stress in the stirrup where it crosses the inclined crack.

Likewise, where the tension reinforcement crosses the crack there is the dowel action, and the resistance offered by the stirrups to the destructive tendency of the dowel action will be discussed later.

†Taub, J., and Neville, A. M., "Resistance to Shear of Reinforced Concrete Beams. Part 1—Beams without Web Reinforcement," ACI JOURNAL, V. 32, No. 2, Aug. 1960 (*Proceedings* V. 57), pp. 193-220.

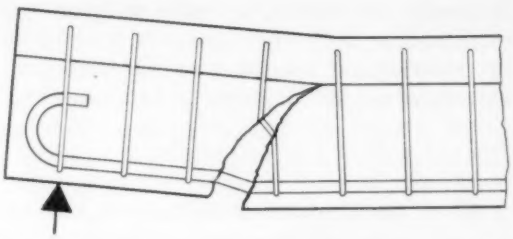


Fig. 15 — Deformation of tension reinforcement after the diagonal tension crack has extended to the level of the tension reinforcement

To determine the magnitude of the components of the force in the stirrup let us consider Fig. 16, representing a free body diagram of the left-hand portion of the beam after the diagonal tension crack has reached the level of the main steel. Equating the horizontal forces, we have:

$$T_{h2} = C' - S_h$$

Also,

$$C' = T_{h1}$$

Hence,

$$T_{h2} = T_{h1} - S_h \quad \text{.....(3)}$$

With increase in the load on the beam the stress in the stirrups increases until it reaches the yield point. This is the beginning of the plastic range, and consequently the stirrups can take practically no further increase in load, although the strain in them continues to increase. The conventional role of stirrups as web reinforcement has thus come to an end and the further behavior of the beam is similar to that of a beam without web reinforcement in which the diagonal tension crack has opened; for this reason a redistribution of internal forces takes place.

Since from the time when the yield takes place the force in the stirrups remains constant it follows from Eq. (3) that any increase in T_{h1} results in an equal increase in T_{h2} and consequently the latter force becomes considerably higher than would be calculated by the elastic theory.

It should be noted that *the yielding of the stirrups does not cause the failure of the beam.* Although the further behavior of the beam

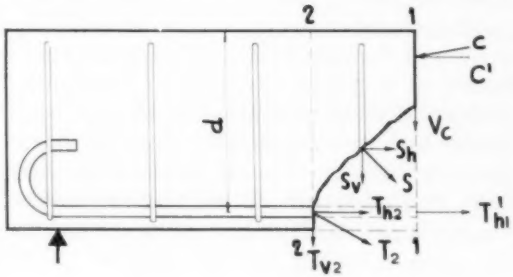


Fig. 16—Free-body diagram after the crack crossing one stirrup has reached the level of the tension reinforcement

is similar to that of a beam without web reinforcement, there is a quantitative difference since in a beam with vertical stirrups a part of the diagonal tension force is carried by the stirrups. The relevant equations can be obtained by reference to Fig. 16.

$$\left.\begin{aligned} T_{v2} &= V - V_c - S_v \\ T_{h2} &= C' - S_h \end{aligned}\right\} \quad (4)$$

When the diagonal tension crack encounters more than one stirrup, S_v and S_h would be sums of the values for the stirrups crossing the crack.

From the above equations it can be seen that in a beam with vertical stirrups the forces T_{v2} and T_{h2}—which, as will be shown later, may be responsible for the collapse of the beam—are smaller than the corresponding forces in a beam without web reinforcement, under the same applied load.

There is one further difference between the behavior of beams with and without vertical stirrups after the redistribution of internal forces has commenced: in the latter case the splitting of concrete along the main reinforcement progresses unopposed up to the support and leads to the collapse of the beam; on the other hand, in a beam with vertical stirrups horizontal splitting cannot take place as it is restrained by the stirrups.

This is the second role of the stirrups in resisting shear failure of a reinforced concrete beam: the composite action of the concrete and the reinforcement is maintained in the part of the beam between the diagonal tension crack and the near support, and the beam can resist an increase in the applied load beyond that which caused the yielding of the stirrups. As long ago as 1935, Evans[26] observed that "provision of web reinforcement preserves the homogeneity of a beam in flexure at higher loads."

The increase in the magnitude of T_{h2} produces a slip of reinforcement which extends progressively toward the support and thus the increase in the tension force in the reinforcement extends toward the support too. But even when the slip of the tension reinforcement has extended up to the end of the beam its load carrying capacity has not been exhausted; owing to the upward pull of the stirrups on the main reinforcement there is a considerable friction between this reinforcement and the concrete, and thus the beam is able to carry an additional load.

A proof of the progressive slip of the main reinforcement was obtained as long ago as 1922, when Richart[28] observed that in rectangular and T-beams (No. 224.1 and 227.1) with corrugated round tension steel and plain round vertical stirrups the measured strains in the tension steel in the outer parts of the beams were much higher than the strains calculated according to the elastic theory; on the other

hand in the middle parts of the beams the measured strains corresponded to the calculated values.

Simultaneously with the increase in the force in the tension reinforcement, and to keep pace with this increase, the compressive force in concrete increases also between the diagonal tension crack and the support. Since the increase in tension occurs, owing to slip, at a greater rate than the increase in the applied moment, the lever arm of the internal forces decreases. This progressive change in force distribution up to the supports results in the beam behaving like a two-hinged tied arch, even when well reinforced in shear. Evans[7] found experimentally that the horizontal stress in the beam has become entirely compressive along certain vertical sections, while at other sections the compressive stress has changed into tension. The tied arch action occurs also in beams without web reinforcement and was described in more detail in Part 1 of this paper.

An experimental verification of the behavior of the stirrups can be obtained from an examination of some test results of Moody, Viest, Elstner, and Hognestad,[3] who measured the strain in stirrups of simply supported beams subjected to two-point loading. For instance, in Beam III-30 (which had a 0.52 percent web reinforcement) no strain in the stirrups was recorded until the appearance of the first diagonal tension crack at a load of 40 kips. With further increase in load the strain in the stirrups increased approximately uniformly until yield under a load of about 130 kips. From then on, the rate of increase in strain in the stirrups increased considerably, and failure of the beam occurred under a load of 215 kips, thus showing that the yielding of the stirrups had by no means exhausted the load carrying capacity of the beam.

TYPES OF SHEAR FAILURE

If the load on the beam is increased beyond the yield of stirrups, failure may occur by flexure at midspan or in shear in one of three ways, *viz:*

(a) By crushing of the concrete at the top of the diagonal tension crack

(b) By the destruction of the tension zone between the lower end of the diagonal tension crack and the beam support

(c) By the opening of a flat-slope crack up to the top surface of the beam

These types of failure are the same as those encountered in beams without web reinforcement but they occur at higher loads. However, failure in flexure after the redistribution of internal forces has started does not generally occur in beams without web reinforcement where the opening of a diagonal tension crack is nearly always followed by failure in shear. The flexural failure deserves, therefore, a more detailed description.

After the diagonal tension crack has developed and the stirrups have yielded, the tension force in the main steel at the intersection with the crack increases greatly in magnitude, and in the case of shear-tension failure it is this force that causes the failure of the beam: the slip of the tension steel at the bottom of the diagonal tension crack extends toward the support. The diagonal tension crack extends also upwards thus reducing the depth of the compression zone. In the case of shear-compression failure crushing of concrete at the upper end of the crack takes place. We can see then that once the redistribution has started, the shear strength of a beam depends on the strength of the compression zone above the diagonal tension crack and of the tension zone at the lower end of the crack. If both of these are strong enough the beam may fail in flexure at midspan, despite the apparently insufficient web reinforcement which resulted in the yield of stirrups. It is important to note here that deformed bars, in the presence of stirrups, provide a stronger tension zone than plain bars. This subject will be discussed in more detail in Part 5 of this paper.

An example of beams with deformed bar reinforcement failing in flexure following the redistribution of internal forces is given by Beams C2-3, D1-2, D2-3, and D2-4, with welded end anchorage plates, tested by Clark.[16] A similar effect can be seen in some tests of Gaston, Siess, and Newmark.[24] For instance, Beam T2Ma, with two ¾-in. diameter deformed bars without hooks or stirrups, loaded at third points, failed in shear under a load of 18.46 kips. A similar beam, T1Ha, *with* ⅜-in. diameter stirrups at 3½-in. centers, failed in tension under a load of 23.4 kips. Although the latter beam was made of 5880-psi concrete as compared with 4320-psi concrete of Beam M2Ha it was undoubtedly the presence of the stirrups that enabled the beam to develop its full strength in flexure. Similar behavior was shown by Beam 27 tested by Larsson[29] (described in Table 10).

If plain bars are used their slip may spread as in beams without stirrups, and the hooks destroy the beam ends. It is possible, however, to restrain the damaging effect of the hooks by binding them so as to prevent their opening. In that case the beam may fail by flexure. The action of the hooks will be discussed in detail in Part 5.

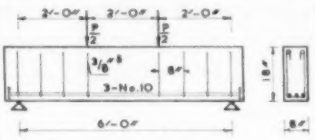
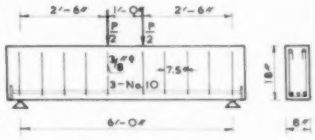
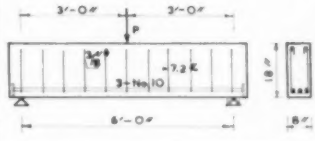
FACTORS INFLUENCING SHEAR FAILURE

Which of the three modes of failure listed above will occur in a particular beam depends on a number of factors; these will now be discussed in turn.

Ratio of shear span to effective depth

The primary influence of the value of this ratio, known as the a/d ratio, is in its effect on the type of failure which can occur in a beam

TABLE 6—INFLUENCE OF a/d RATIO ON SHEAR STRENGTH OF BEAMS WITH VERTICAL STIRRUPS

Investigator	Beam No.	BEAM DETAILS	$\frac{a}{d}$	f'_c psi	p percent	r percent	P_u kips
Clark[16]	C4		1.56	3550	3.1	0.34	139
	B1		1.95	3490	3.1	0.37	120
	A1		2.34	3500	3.1	0.38	101

with a given reinforcement: when the value of a/d is small shear-compression failure takes place, in a manner similar to that described in Part 1 of this paper.

When a/d is larger shear-tension failure occurs, again as described in the case of beams without web reinforcement, except that the failing load is considerably higher.

The influence of the a/d ratio can be seen from the test results of Clark's[16] assembled in Table 6; the photographs of some of the authors' tests (Fig. 17, 18, and 19) show the different modes of failure: R33—shear-compression; R42—balanced shear-compression and shear-tension; and T33—shear-tension. The relevant data are given in Table 7.

Wilby[17] and also Ohno and Arakawa[30] confirmed the increase in the strength of a beam with the decrease in the a/d ratio for various values of r. Since this relation is not linear but in the form of a hyperbola it follows that the shear strength of a beam depends not only on the shearing force applied but also on the magnitude of the bending moment.

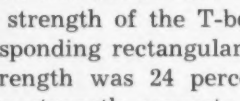
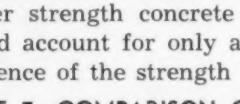
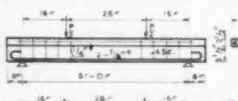
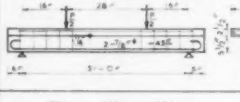
It may be interesting to note that while the influence of the a/d ratio on the shearing force at failure is considerable, Ohno and Arakawa[30] believe that the load at which the first diagonal tension crack occurs is practically independent of the value of a/d.

Shape and proportions of beam

It is believed that no systematic comparison of the shear strength of rectangular, T-, and L-beams has so far been made, and, as mentioned in Part 1 of this paper, the various codes consider the shear strength of T- and L-beams to be no greater than that of a rectangular beam of the same depth and of breadth equal to the breadth of the web of the T- or L-beam. To verify this assumption an experimental investigation has been conducted by the authors at the University of Manchester. In the series of tests on beams with vertical stirrups three rectangular, three T-, and two L-beams were used. Within each group the beams were similar, the flanges being the only difference. The reinforcement consisted of plain round bars with end hooks. The test details were given in Part 1 and the results are given in Table 7.

From this table it can be seen that for the a/d ratio of 2.37 the ultimate strength of the T-beams was 57 percent higher than that of the corresponding rectangular beam. For the a/d ratio of 3.41 this increase in strength was 24 percent. The T-beams were made of somewhat higher strength concrete than the rectangular beams but this alone would account for only a small increase in the ultimate strength; the influence of the strength of concrete was discussed in detail in Part 1.

TABLE 7—COMPARISON OF SHEAR STRENGTHS OF RECTANGULAR, T-, and L-BEAMS

Beam No.	BEAM DETAILS	$\frac{a}{d}$	f_{cu} (+) psi	p percent	r percent	P_u kips	v_u psi	$\frac{v_u}{f_{cu}}$	Mode of failure
R 33		2.37	3600	4.46	0.32	19.7	413	0.115	Shear-compression
T 33		2.37	5720	4.46	0.32	31.0	657	0.125	Destruction of beam end by hooks
L 33		2.37	5180	4.46	0.32	28.6	606	0.117	Destruction of beam end by hooks
R 35		3.41	3900	4.46	0.32	22.8	483	0.124	Flexure failure
T 35		3.41	4990	4.46	0.32	28.2	597	0.120	Destruction of beam end by hooks
L 35		3.41	5120	4.46	0.32	22.0	465	0.091	Flexure failure

Notes: (+) f_{cu} measured on 3" cubes
All steel consists of plain round bars

Likewise, a comparison of the L- and rectangular beams shows an increase in strength of 45 percent for the a/d ratio of 2.37, a part of this increase being due to increased concrete strength. For the higher a/d ratio both beams failed in flexure.

These data are believed to be of interest in view of the requirements of the ACI, British, German and, indeed, all other codes, as mentioned at the beginning of this section.

Not only the ultimate strength but also the mode of failure of the beams tested are of interest. The rectangular beam, R33, failed in shear-compression, while the corresponding T-beam, T33, failed in shear-tension (Fig. 17). The actual collapse was caused by the destruction of the beam end by the hooks of the tension steel, thus showing the influence of the compression flange in preventing the crushing of the concrete above the diagonal tension crack, and in increasing the load carrying capacity of the beam. In the beams with a/d ratio of 3.41 the rectangular beam, R35, failed in flexure-compression while the corresponding T-beam, T35, failed in shear-tension. The crack patterns of these beams at failure are shown in Fig. 18.

No comparisons of simply supported rectangular and T-beams seem to be available in the literature but in the case of restrained beams Elstner, Moody, Viest, and Hognestad[3] give data on two pairs of beams. Their reinforcement consisted of deformed bars with hooks and vertical stirrups of ⅝-in. diameter at 6-in. centers. The T-beams (I-16a, b) differed from the rectangular beams (I-12a, b) only by the presence of the flange.

The average cracking load was 40 kips for the rectangular and 60 kips for the T-beams. The corresponding figures for the ultimate loads were 175 and 275 kips, respectively. It seems, therefore, that the provision of the flange increased both the cracking and the ultimate load by about one-half.

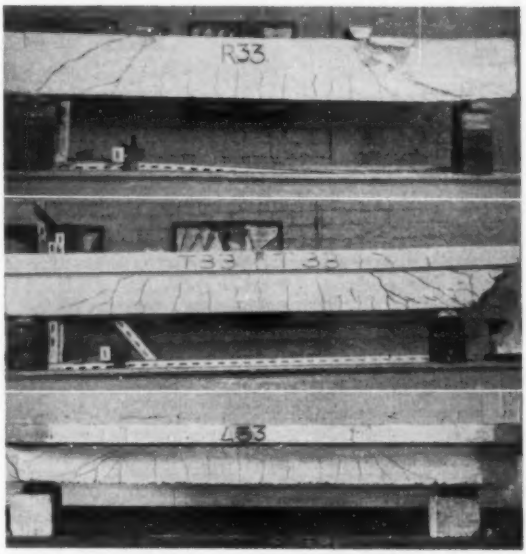


Fig. 17—Beams of Series 33: rectangular (top), T-beam (middle), and L-beam (bottom)

Percentage area and spacing of vertical stirrups

From Table 8 it can be seen that the higher the percentage area of web reinforcement, r, the higher the load carrying capacity of the beam. This is, indeed, common knowledge, but it is important to note that the increase in this capacity is small and, in particular, is not proportional to the total amount (weight) of the stirrups. This is so because the ultimate resistance of a beam to shear is only in part due to the stirrups, and the beam can take considerably more load after they have yielded. This aspect of the action of stirrups is shown, for instance, by a comparison of two beams tested by Moody, Viest, Elstner, and Hognestad[3] which differed only in the cross-sectional area of vertical stirrups: Beam III-30 had $r = 0.52$ percent, while in Beam III-31 $r = 0.95$ percent. The corresponding values of the ultimate load were 215 and 228 kips, respectively—a negligible increase for almost a doubling of the weight of the stirrups. An explanation of this behavior can be obtained from a study of the stirrup strains of the two beams plotted in Fig. 20. In both beams the stirrups show practically no strain up to a load of 40 kips. Above this load the stirrup strains in both beams increase considerably, from which it could be inferred that the first diagonal tension crack has opened. The cracking load reported by the investigators is 40 kips for each beam. Considering now the stirrup with the highest strain in each beam, it can be seen that up to a load of 100 kips the stirrup strains in both beams are the same. This means that the width of the diagonal tension crack, where it intersects the stirrup, was the same in both beams, notwithstanding the considerable difference in the stirrup diameter. Above the load of 100 kips, however, the stirrup strains in Beam III-30 increase at a greater rate than in Beam

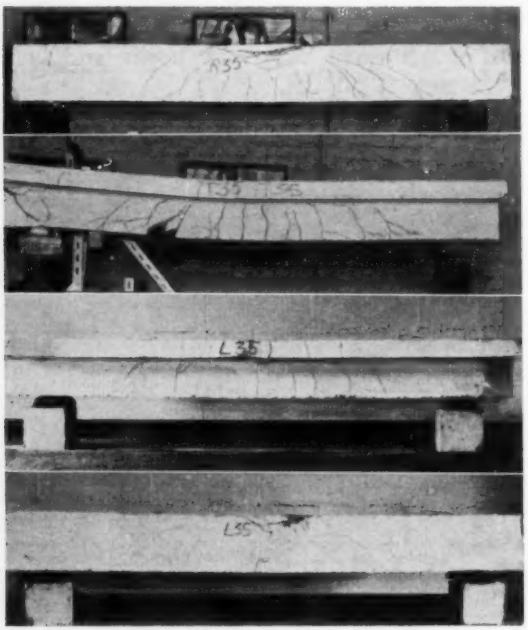


Fig. 18—Beams of Series 35: rectangular (top), T-beam (second from top), and L-beam (two bottom photos)

Fig. 19—Balanced shear-compression and shear-tension failure

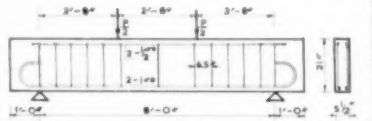
III-31 until yield point is reached at 123 and 150 kips, respectively.

A redistribution of internal forces now takes place and the further *increase* in the load carrying capacity depends only on the strength of the compression zone of the beam at the upper end of the diagonal tension crack and of the tension zone at its lower end. Since the beams considered differed only in their web reinforcement, their compression and tension zones were similar and the load increments between the yield of the stirrups and the ultimate should be the same; the actual values are 92 and 78 kips, respectively.

A similar relation between the cross-sectional area of the stirrups and the ultimate strength of the beam can also be seen from some of Moretto's test data[31] on beams with welded stirrups given in Table 8. In his tests, the *increase* in the shear capacity of the beam beyond the yield of stirrups was found to be directly proportional to the compressive strength of concrete. Since the tension steel and anchorage were very strong and were similar in all beams it was the compression zone that governed the behavior of the beam, once the redistribution of internal forces had started.

These observations on Moretto's and Moody, Viest, Elstner and Hognestad's test results, are believed to confirm the postulated redistribution

TABLE 8—INFLUENCE OF PERCENTAGE AREA OF VERTICAL STIRRUPS ON SHEAR STRENGTH OF RECTANGULAR BEAMS

Investigator	Beam No.	BEAM DETAILS	f'_c psi	p percent	Size of stirrups	r percent	p_u kips
Moretto[31]	I-V 1/4		3460	4.00	1/4" Φ	0.28	117
	I-V 3/8		3115	4.00	3/8" Φ	0.615	147
	I-V 1/2		3665	4.00	1/2" Φ	1.096	157
	Ia-V 1/4		3540	1.88	1/4" Φ	0.28	106
	Ia-V 3/8		3335	1.88	3/8" Φ	0.615	117
Moody, Viest, Elstner and Hognestad[3]	III-30		3680	4.25	#3	0.52	215
	III-31		3250	4.25	#4	0.95	228

of internal forces in a beam with vertical stirrups.

Referring again to Fig. 20 it may be observed that the stirrup stresses calculated for the ultimate load on the basis of the current codes are:

(a) According to ACI Code:	142,500 psi	for Beam III-30
	86,200 psi	for Beam III-31
(b) According to British Code:	159,500 psi	for Beam III-30
	95,800 psi	for Beam III-31

The divergence between these figures and the actual behavior of stirrups is evident.

The results of some tests on T-beams with vertical stirrups, performed by Bach and Graf,[32] and have been collected in Table 9 to show even more clearly that the use of the truss analogy for the calculation of stirrup size and spacing leads to incorrect results. According to this method of calculation, the higher the percentage of web reinforcement, the higher the load carrying capacity of the beam, provided, of course, the beam is sufficiently strong in flexure. Now, comparing the two extreme cases of Table 9—the highest and the lowest areas of reinforcement—an anomaly becomes apparent: for Beam b with r of 1.56 percent, the ultimate load was 94.4 kips, and the stress in the stirrups, calculated by the elastic theory, 27,100 psi. For Beam 13 with r of 0.13 percent, the ultimate load was 72.4 kips, and the calculated stress in stirrups 290,000 psi. Such a stress is clearly impossible.

It is interesting to observe that while the weight of the stirrups in Beam b was 11 times the weight of the stirrups in Beam 13, the ratio of the corresponding ultimate loads was only 1.3. A similar comparison is afforded by Beams 15 and b: doubling the weight of stirrups produces no effect on the ultimate strength of the beam. This shows how uneconomical is the use of large diameter stirrups.

The explanation of this anomaly lies in the fact that the stirrups carry only a part of the shearing force applied, and after they have

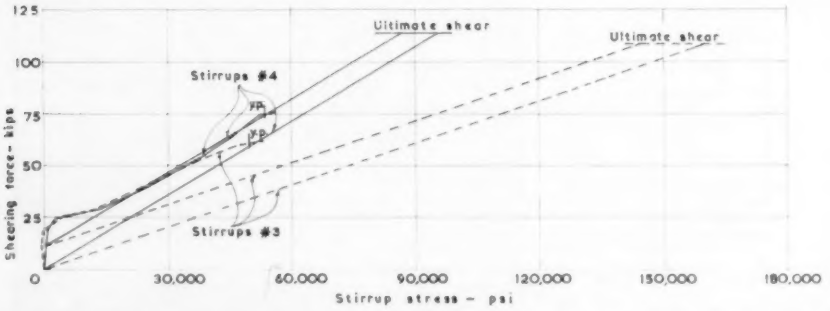
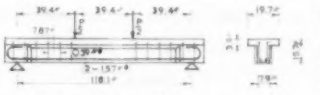
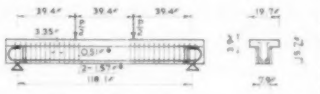
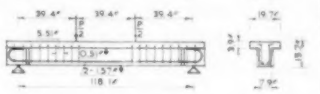
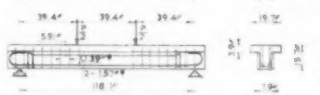
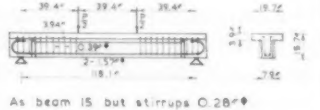
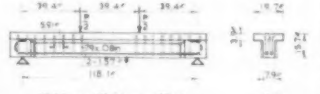
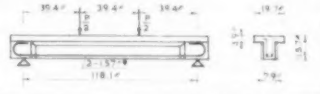


Fig. 20—Shearing force against stirrup stress for Moody, Viest, Elstner, and Hognestad's[3] Beam III-30 (with 3/8-in. diameter stirrups) and Beam III-31 (with 1/2-in. diameter stirrups). Straight lines represent stirrup stress according to current codes: ACI (upper line), British (lower line)

yielded the further increase in the shear capacity of the beam does not depend on the web reinforcement. The contribution of stirrups reaches a maximum when yield occurs.

Let us analyze now some of the beams of Table 9 on the basis of the current codes. According to the British Code the allowable shearing stress in concrete would be 100 psi and the stirrup stress 20,000 psi. For Beam 13 the shearing resistance of concrete would be the criterion since the shearing resistance of the stirrups is the lesser of the two, and they are *not considered additive*. Thus the design load would be 18.5 kips, and the factor of safety $72.4/18.5 = 3.9$.

TABLE 9—INFLUENCE OF SPACING AND PERCENTAGE AREA OF VERTICAL STIRRUPS ON SHEAR STRENGTH OF T-BEAMS

Investigator	Beam No.	BEAM DETAILS	r percent	P_s kips	P_u kips	v_u psi	Stirrup stress according to British and German Codes psi
Bach and Graf[32]	8		0.39		80.0	412	106,000
	b		1.56	66.2	94.4	490	27,100
	c		0.95	59.5	86.0	446	40,700
	11		0.52	52.9	82.9	431	82,500
	12	As beam 11 but stirrups 0.28"ϕ	0.26	59.5	79.4	412	160,800
	13	As beam 11 but stirrups 0.20"ϕ	0.13	52.9	72.4	371	290,000
	15		0.79		94.1	489	62,500
	16	As beam 15 but stirrups 0.28"ϕ	0.39	52.9	88.2	457	119,100
	17	As beam 15 but stirrups 0.20"ϕ	0.20		80.1	412	213,000
	21		0.53	61.7	87.5	448	85,000
	7		0	48.5	54.5	285	

Notes: All data, except b and c, are averages for three similar T-beams.
All steel consists of plain round bars.
Average concrete strength $f'_{cu} = 3521$ psi

In the case of Beam b the permissible stirrup stress would be reached under a load of 57.4 kips. Hence, the factor of safety is 94.4/57.4 = 1.65, that is *less than* 2.

The incorrectness of these calculations is apparent. Furthermore, it should be observed how fallacious is the belief that the provision of shear reinforcement consisting of vertical stirrups according to the current British, German, and many other codes is a safeguard against failure of a beam in shear.

The requirements of the ACI Code are more rational as the use of web reinforcement consisting of vertical stirrups only is not permitted where the shearing stress exceeds 0.08 f_c' or 240 psi. Since the shearing resistance of the stirrups is added to that of concrete, Beam 13 would have a working load of 21.7 kips, giving a factor of safety of 3.3.

For Beam b the shear capacity would be limited by the shearing stress of 240 psi and thus its working load would be no greater than that of Beam 15, *viz.*, 44.5 kips. The actual failing loads of these two beams are practically the same, and give a factor of safety of 2.1.

It should be emphasized that all the beams reported in Table 9 failed in shear-tension. Regardless of the size and spacing of the stirrups the collapse of all these beams was caused by the destruction of the beam end by the hooks of the highly stressed tension steel. Beam 7, included for comparison, had no web reinforcement, and here the destruction of the beam end was preceded by the pressing down of the main steel at the bottom of the diagonal tension crack and splitting of the beam along the main steel.

Referring now to the load under which the slip of the main steel at the beam end was first observed, a basic difference between beams without web reinforcement and beams with stirrups can be noted. While in beams with stirrups the slip load is only somewhat higher than in beams without web reinforcement, the increase in load *beyond* the slip load is considerable. On the other hand, in beams without web reinforcement the slip is soon followed by the ultimate failure.

In this connection Beam 21 should be discussed in detail. The stirrups are of the multiple type (particularly common in France) so that there is a separate stirrup passing round each bar of the main reinforcement. This type of stirrup would be expected to be more effective in resisting the pressing down of the main steel and both the slip and ultimate load would be increased. This is clearly borne out by a comparison of Beams 21 and 11, whose percentage areas of stirrups were equal. Multiple stirrups are particularly important in beams with more than three main bars in one layer in a wide web when a single stirrup cannot successfully resist the pressing down of the center bars.

This function of stirrups is not apparent in beams with a very small a/d ratio since the tendency of the concrete to split along the tension

steel is absent.

It is important to observe that the action of the stirrups in preventing the pressing down of steel depends more on their layout than on their size or, within limits, on their spacing.

The spacing of the stirrups is, however, of paramount importance in resisting shear failure of a beam. The greater the number of stirrups crossed by a crack, the less it opens and the later occurs the yield of the stirrups, and hence the higher the ultimate load on the beam. Thus once again it can be seen that the use of the truss analogy, which assumes the size and spacing of stirrups to compensate one another, is not correct.

The influence of the spacing of the stirrups can be seen by comparison of the test results of Beams c and 16. Although the latter beam has a lower percentage of web reinforcement, it has a higher ultimate load owing to the smaller spacing of the stirrups. Similar effects can be seen by comparing Beams 16 and 11. Stirrups are effective only if crossed by a diagonal tension crack or a potential crack, and they have, therefore, to be spaced in such a manner that each diagonal tension crack would encounter at least one stirrup between the level of the main steel and the neutral axis; only then would the compression zone be unimpaired. This means that the distance between two adjacent stirrups must not exceed $d(1 - k)$.

Comparing Beams 8 and 16, it can be seen that they have the same percentage area of web reinforcement, but the spacing of stirrups in the former is twice the spacing in the latter beam—$d(1 - k)$ as compared with $\frac{1}{2}d(1 - k)$—the cross-sectional area of the stirrups being proportional to the spacing. The ultimate load of Beam 16 was 10 percent higher than that of Beam 8.

The influence of the spacing of stirrups was confirmed by Sunderland[27] who tested a series of beams with a constant r of 1 percent. When the stirrup spacing was greater than $\frac{1}{2}jd$, the diagonal cracks were wider and less numerous and the failing loads lower than at smaller spacings. When the stirrup spacing exceeded 1.06 jd, Sunderland[27] observed "a material reduction in stirrup stresses owing to the lack of effectiveness."

But even very wide spacing of stirrups is not wholly ineffective as their presence increases the load carrying capacity of the beam compared with that of a beam without web reinforcement.[17] This is so because vertical stirrups resist the spreading of splitting along the tension steel; also, for small a/d ratios, the crack going straight from the load point to the support is likely to encounter a stirrup in its path. This influence of stirrups is borne out by Smith's[25] results on beams with the a/d ratios of 2, 3, and 4.4, given in Table 10.

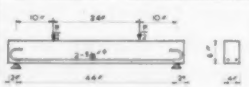
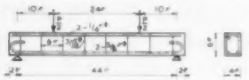
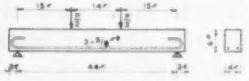
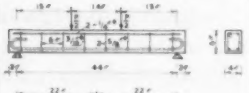
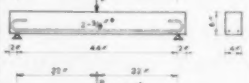
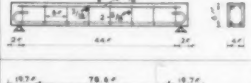
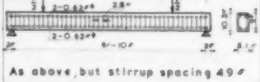
Stirrups are usually made of mild steel and, as shown by some tests of Larsson's,[29] this type of steel is preferable to high tensile steel.

Cold-drawn steel becomes brittle due to cold-hardening, and is not suitable for bending. Larsson observed cracks, which could not be easily detected by a naked eye, in the corners of cold-drawn stirrups, and beams made with such stirrups failed prematurely and suddenly in shear. Furthermore, mild steel stirrups, owing to the lower permissible stress in such a steel, have to be more closely spaced than stirrups made of high quality steel, and the advantages of such close spacing in reducing the risk of local shear failure have been discussed.

Percentage area and arrangement of main reinforcement

It has been shown earlier that the yielding of stirrups does not cause the collapse of a beam but only a redistribution of internal forces, with a consequent increase in the force in the main reinforcement. In beams failing in shear-tension the further increase in load on the beam, and therefore its load carrying capacity, depend thus on the percentage area and arrangement of the main reinforcement. Table 11 shows the influence of the percentage steel area, p, for some beams

TABLE 10—TEST DATA ON THE INFLUENCE OF STIRRUPS AT SMALL AND LARGE SPACINGS ON SHEAR STRENGTH OF RECTANGULAR BEAMS

Investigator	Beam No.	BEAM DETAILS	$\frac{a}{d}$	f_{cu} psi	p percent	r percent	P_u kips	v_u psi	$\frac{v_u}{f_{cu}}$	$\frac{r}{d(1-k)}$	Mode of failure
Smith[25]	20 AB		2.0	4700	3.07	0	41.1	1178	0.250	—	Shear
	15 AB		2.0	4580	3.07	0.23	54.0	1545	0.337	2.0	Shear
	19 AB		3.0	4575	3.07	0	25.8	740	0.162	—	Shear
	16 AB		3.0	4750	3.07	0.23	38.2	1092	0.230	2.0	Shear
	18 AB		4.4	5090	3.07	0	21.3	610	0.120	—	Shear
	14 AB		4.4	5680	3.07	0.23	28.7	823	0.145	2.0	Balanced flexure-shear
Larsson[28]	27		2.2	3370	1.31 (*)	1.09	51.6	637	0.189	0.5	Flexure-failure
	1	As above, but stirrup spacing 49"	2.2	2850	1.31	0.62	40.8	503	0.177	0.8	Shear-tension

(*) High tensile deformed bars

of Moretto's[31] and Clark's,[16] and it can be seen that the greater the value of p the higher the load carrying capacity of the beam. Unfortunately, no data are available for beams with a large a/d ratio.

The influence of p is expected on theoretical grounds as the stronger the main tension steel the more it opposes the widening of the diagonal tension crack, and therefore the higher the ultimate load on the beam.

It is interesting to note that the 1949 Russian Code of Practice assumes that a part of the shearing force acting on a beam can be assumed to be carried by the tension reinforcement. In beams subjected to point loading this is taken as 20 percent of the shearing force acting. In beams subjected to a uniformly distributed load, where the shearing stress exceeds the allowable value for concrete only, the proportion of the shearing force assumed to be resisted by the main reinforcement is between 20 and 40 percent, depending on the length of the anchorage provided beyond the inner edge of the support.

A confirmation of the influence of the tension steel on the shear strength of a beam, and also an insight into the mode of redistribution of internal forces, can be obtained from two tests described by Mörsch.[2] The data for the two beams are given in Table 12, the differ-

TABLE 11—INFLUENCE OF PERCENTAGE AREA OF MAIN REINFORCEMENT ON SHEAR STRENGTH OF BEAMS WITH VERTICAL STIRRUPS

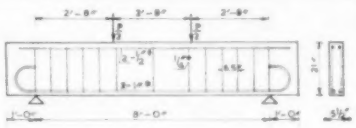
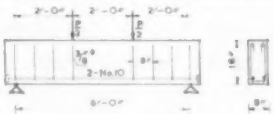
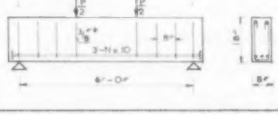
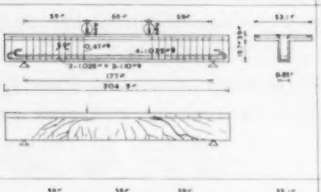
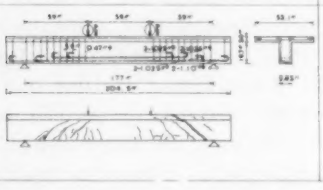
Investigator	Beam No.	BEAM DETAILS	$\frac{a}{d}$	f'_c psi	p percent	r percent	P_u kips
Moretto[31]	Ia - V 1/4		1.64	3540	1.88	0.28	106
	I - V 1/4		1.76	3460	4.00	0.28	117
	Ia - V 3/8	As beam Ia - V 1/4 but stirrups 3/8" ϕ	1.64	3335	1.88	0615	117
	I - V 3/8	As beam I - V 1/4 but stirrups 3/8" ϕ	1.76	3115	4.00	0.615	147
Clark[16]	C1-1,3		1.56	3600	2.07	0.34	118
	C4-1		1.56	3550	3.10	0.34	139

TABLE 12—INFLUENCE OF LAYOUT OF MAIN REINFORCEMENT ON SHEAR STRENGTH OF T-BEAMS WITH VERTICAL STIRRUPS

Beam No	Beam Details	$\frac{a}{d}$	f_{cu} psi	p percent	r percent	P_u kips	v_u psi	$\frac{v_u}{f_{cu}}$	Mode of failure
1132		2.8	1650	3.3	0.60	166	455	0.276	Shear-compression
1133		2.8	1710	3.3	0.60	122	335	0.196	Splitting of beam along diagonal tension crack

ence between them being that in Beam 1132 all the main reinforcement was carried beyond the supports, while in Beam 1133 only one-quarter of the main steel was so carried, the remainder being cut off in accordance with the diminution of the bending moment away from midspan. The actual points of cutoff were, of course, a "bond distance" beyond the theoretical points of curtailment.

From Table 12 it can be seen that the ultimate strength of Beam 1132 was 36 percent higher than the strength of Beam 1133, and there was also a considerable difference in the crack patterns of the two beams. In the former, there were a number of diagonal tension cracks in the shear span and failure occurred by the crushing of the concrete. An inspection of the stirrups after the beam was broken up revealed that the stirrups crossing the main cracks had yielded. In Beam 1133 three diagonal tension cracks opened, each starting at the point of curtailment of the main steel, i.e., where there is a sudden change in the strain in the bottom of the beam.

In Beam 1133 the crack nearest to the support opened wider with the increase in the applied load and this crack eventually led to the collapse of the beam. Here again the stirrups crossing the critical crack had yielded.

In the early stages of loading, i.e., before the yielding of the stirrups, for the same applied load the cracks were less developed in Beam 1132 than in Beam 1133 because in the former beam there was a larger area of main steel resisting the cracks, and the deflection of the beam was smaller. After the stirrups had yielded a redistribution of internal forces started and, although in Beam 1132 the critical crack was further away from the support and, therefore, the force in the tension steel at the end of the diagonal tension crack somewhat greater than

in Beam 1133, the different amounts of reinforcement in the two beams seriously affected further developments. In Beam 1132 there were eight bars of the main reinforcement, as compared with two bars in Beam 1133, and they could therefore offer a higher resistance to the destructive action of the tension force. For this reason Beam 1132 failed in shear-compression while Beam 1133 failed in shear-tension under a considerably lower load. This behavior is believed to confirm the role of the tension steel in the shear resistance of reinforced concrete beams.

From the above experiment it can be also inferred that cutting off of the main steel in the tension zone, practiced in some design offices, lowers the load carrying capacity of a reinforced concrete beam. If the steel is bent up and cut off in the compression zone, the slip of reinforcement is resisted better, and the bent-up steel helps also in preventing the widening of the diagonal tension crack.

Little is known about the influence on shear resistance of the compression reinforcement in a beam. According to Laupa, Siess and Newmark's analysis[20] this reinforcement contributes to the shear strength of all types of beams, but Larsson's tests[29] indicate that the compression reinforcement does not affect the shear capacity of a beam with vertical stirrups. It is possible that the effectiveness of the compression reinforcement in resisting shear depends on there being a sufficient number of well anchored stirrups to restrain the steel buckling. Tests on the shear resistance of beams with varying amounts of compression reinforcement are in progress at the University of Manchester and so far they indicate that this reinforcement does not contribute to the shear strength of a beam.

Compressive strength of concrete

The effects of this variable in beams without web reinforcement were discussed in detail in Part 1 and since they are the same for beams with vertical stirrups they need not be further mentioned. It should be remembered, however, that in the case of shear-compression failure the compressive strength of concrete plays an important role. The experimental results illustrating the influence of the strength of concrete are given in Table 13.

Arrangement of loading

When the load applied consists of one or two point loads the first diagonal tension crack opens usually in the vicinity of the point of application of the load. Further diagonal tension cracks open in the shear span but generally the crack pointing toward the load is eventually responsible for the failure of the beam. This can be seen clearly from the photographs of the beams tested by the authors: R33, R42, and T33 (Fig. 17 and 19), and also from an inspection of test data of

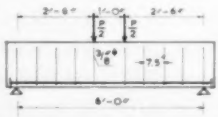
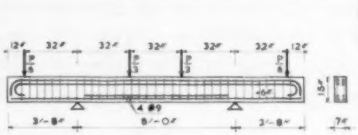
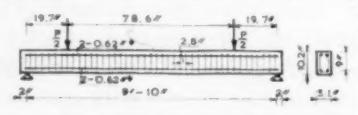
Clark[16] and Moody,[3] for rectangular beams, and Bach and Graf[32] for T-beams.

When the load applied is uniformly distributed a number of diagonal tension cracks form, usually as a development from flexural cracks. One of these diagonal tension cracks in the part of the beam somewhat removed from the supports, i.e., not in the zone of maximum shearing force, widens with the increase in the load applied and leads to the failure of the beam, as shown by tests of Bernaert and Siess[10] and Bach and Graf.[22] This behavior was discussed in some detail in Part 1 of this paper.

CONCLUSIONS

From the test data reviewed in this paper, it can be seen that in many cases vertical stirrups alone cannot give full protection from shear failure, although this would be expected from the British and German codes. Shear failure can, in fact, take place and its mode is usually similar to that of beams without web reinforcement but, owing to the presence of stirrups, the ultimate load is considerably higher.

TABLE 13—INFLUENCE OF COMPRESSIVE STRENGTH OF CONCRETE ON SHEAR STRENGTH OF BEAMS WITH VERTICAL STIRRUPS

Investigator	Beam No.	BEAM DETAILS	a/d	f'_c psi	p percent	p' percent	P_u kips
Clark[16]	C 4		1.56	3550	3.1	0.34	139
	C 6		1.56	6650	3.1	0.34	193
	C 3		1.56	2020	2.07	0.34	92
	C 1		1.56	3805	2.07	0.34	126
	B 1		1.95	3490	3.1	0.37	120
	B 6		1.95	6110	3.1	0.37	171
Moody, Viest, Elstner and Hognestad[3]	I-10b		2.67	2810	4.76	0.52	138
	I-10a		2.67	3070	4.76	0.52	163
	I-11b		2.67	3180	4.76	0.95	174
	I-11a		2.67	3560	4.76	0.95	190
	I-12b		2.67	3220	4.76	1.47	159
	I-12a		2.67	4000	4.76	1.47	190
Larsson[28]	23		2.17	1735 (+)	1.31 (x)	1.09	39.7
	27		2.17	3370	1.31	1.09	51.6
	28		2.17	3970	1.31	1.09	57.4

Notes: (+) f_{cu}
(x) High tensile deformed bars

Furthermore, when stirrups are provided, the opening of the initial diagonal tension crack is followed by formation of other diagonal cracks and by a considerable increase in the applied load before failure occurs. Another difference between beams without and with stirrups is that in the latter case a sudden collapse of the beam does not take place. This fact is of tremendous importance and would warrant the provision of web reinforcement in *all* beams regardless of the value of the nominal shearing stress; should the beam be accidentally overloaded or subjected to a load distribution different from that assumed in design, the provision of stirrups would prevent failure without warning from taking place.

It has been shown that the resistance to shear of a reinforced concrete beam depends to a considerable degree on the strength of the compression and tension zones in the vicinity of the diagonal tension crack. Little is known about this from the point of view of the shear resistance, and it is, therefore, not practicable to suggest general formulas for the shear strength of any beam, but some recommendations on the arrangement of stirrups can be made on the basis of the research work done.

To ensure that *any* potential diagonal tension crack encounters a stirrup and does not, therefore, open excessively and reduce the depth of the compression zone, the stirrups should be spaced at a distance not greater than $d(1-k)$. For higher values of the nominal shearing stress, this spacing should be smaller, preferably $\frac{1}{2}d(1-k)$.

The role of stirrups as web reinforcement is limited because after they have yielded they can take no further increase in load.

It has been proved experimentally that an increase in the cross-sectional area of stirrups results in only little increase in the load carrying capacity of the beam. The reason for this is that the role of vertical stirrups in resisting shear is twofold: they carry a part of the diagonal tension, and also prevent the splitting of the beam at the level of the tension steel. This latter action is achieved by the stirrups practically regardless of their size, and as far as the resistance to the diagonal tension is concerned, the stirrups are only one of the components of the beam resisting this tension. Thus it would appear that the provision of stirrups of large cross-sectional area is uneconomical: the gain in strength is low relative to the increased cost of the material and the effort in its bending and fixing. However, sufficiently close spacing of stirrups is essential.

Furthermore, it can be seen that the code formulas for the size and spacing of vertical stirrups are not based on the correct interpretation of their action in a reinforced concrete beam—heavy stirrups do not guarantee a satisfactory resistance to shear.

It is important, however, to arrange the stirrups so that they can effectively prevent the pressing down of the main steel and the con-

sequent splitting of the beam at the level of that steel. For that reason in wide beams with more than three bars in one layer, multiple stirrups should be used.

The main tension steel contributes materially to the shear resistance of a beam. It has been shown that cutting off of such steel lowers the load carrying capacity of the beam and, by introducing a sudden change in strain in the steel, aggravates the formation of diagonal tension cracks. It seems therefore advisable for the main steel, when no longer required to carry the flexure tension, to be bent up and anchored in the compression zone. In this manner the inclined bars resist the opening of diagonal tension cracks and thus act as additional web reinforcement; the anchoring of the tension steel in the compression zone has also the effect of improving the composite action of the beam. The advantages of bending-up the main steel are discussed further in Part 3 of this paper.

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*References are numbered consecutively beginning with Part 1 of this five-part series.

Received by the Institute Apr. 28, 1959. Title No. 57-15 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 3, Sept. 1960 (Proceedings V. 57). Separate prints are available at 60 cents each.

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Discussion of the entire series (five parts) should reach ACI headquarters in triplicate by Mar. 1, 1961, for publication in the June 1961 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Smooth-riding bridge decks

NOMER GRAY, *Bulletin* No. 243, Highway Research Board, Mar. 1960, pp. 18-27

Presents some of the distinctions between the construction of concrete bridge decks and construction of concrete pavement on grade. The factors that affect smooth riding are examined in detail and the need for good construction practices and close supervision is emphasized. The report is confined to a discussion of bridges with concrete decks supported on steel members and to single course pavements in which the structural slab and wearing surface are placed monolithically.

Bridge decks repaired under express traffic

Roads and Streets, V. 103, No. 5, May 1960, pp. 127-129, 138

Major deck repairs involving novel suspended forms were made on expressway bridges. Parts of the deck and T-beams of these structures had deteriorated but it was necessary to maintain traffic on the expressway as well as on the underpass roadway below.

After the deteriorated areas were cut out (some of it all the way through the slab) the forms were erected. Forms for the underside of the slab were held in position with form bolts. The bolts extended up through the gaps in the deck and through lines of paired 2 x 8-in. walers. The walers were supported on heavy timbers blocked up off of undisturbed pavement areas so as to leave a clear space for the concreting.

Construction

Main building of the boiler water pretreatment plant of the Kombinat Schwarze Pumpe — A multistory building of prefabricated concrete elements (Das Hauptgebaeude der Speisewasseraufbereitung im Kombinat Schwarze Pumpe—ein Stockwerksgebaeude aus stahlbetonfertigteilen)

OTTO PLATZER, *Bauplanung-Bautechnik* (Berlin), V. 13: No. 8, 9, and 10; Aug., Sept. and Oct. 1959; pp. 357-363, 417-420, and 465-470

Reviewed by J. F. LEPPMANN

A detailed description of the 350 ft long, 102 ft wide, and 106 ft high structure which — with the exception of one end bay, the so-called head house — was built entirely of precast concrete elements. While the installed equipment is of a wide variety of sizes and weights (maximum is about 310,000 lb per unit) relatively few basic prefabricated shapes were used. All columns have the same rectangular section, about 3 x 2 ft; they differ from each other only in the number and location of the cast-on brackets which support the floor and wall elements. All floor elements have the same area, fitting into the basic grid module of 18 ft 6 in. x 25 ft; each element has

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A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 3, Sept. 1960, *Proceedings* V. 57. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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two intermediate floorbeams and L-shaped edge beams, varying in depth with the load they carry. Quasi-monolithicity was produced by filling the space between the L-shaped beams with cast-in-place concrete. Structural continuity was achieved by either lapping or welding projecting reinforcing bars. All precast elements were steam cured between 550 and 600 C-hr. Two rail born whirly cranes, each with a capacity of about 20 (long) tons at a 30 ft radius, were used to erect the structure.

Folded plate concrete roof for octagonal house

Progressive Architecture, V. 41, No. 5, May 1960, pp. 188-189

An octagonal house in Florida is roofed by eight tapered concrete plate sections, each being 24 ft wide and 34 ft long.

By combining the advantages of thin shell design, multiuse of prefabricated forms, and lift-slab construction, the architect achieved a low cost concrete roof and proved that medium budget construction can benefit from these advantages. The 3200 sq ft area of concrete roof was cast and erected at a cost of $2.25 per sq ft, including supporting columns.

Shell vault of the Exposition Palace, Paris

NICOLAS ESQUILLAN, *Proceedings*, ASCE, V. 86, ST1, Jan. 1960, pp. 41-70

AUTHOR'S SUMMARY

General requirements for the structure are given. The preliminary and final designs are described and the function of structural elements is explained. Principles of design, strength of materials, allowable stresses, and forces affecting the roof design are presented. The investigation of local and general buckling of the shells, the influence of creep and other factors on the stability of the shell vault are discussed in detail. Research studies are described. The solution of numerous construction problems to meet design requirements are also presented.

Naturbetong — A common ground for the architect and artist

Concrete Quarterly, Cement and Concrete Association (London), No. 44, Jan.-Mar. 1960, pp. 13-21

Describes a Norwegian method of finishing concrete called "Naturbetong" (or natural concrete) which exposes a maximum amount of aggregate. The method might be described as a refinement of the intrusion method of placing concrete coupled with sandblasting. The first stage is to fill the formwork with washed and graded aggregate of over 5/8 in. size. The aggregate is consolidated and the mortar is injected. After removal of the forms, the surface mortar is removed by sandblasting. By varying the depth of sandblasting or by leaving some parts unblasted in regular or irregular patterns, a whole range of treatments becomes possible.

Two buildings on which the process has been used are described, a government building and a church in Oslo.

Construction Technique

13th German industrial fair in Hannover (13. Deutsche Industriemesse in Hannover)

P. KLUTH, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 4, Apr. 1959, pp. 116-127

Reviewed by FERDINAND S. ROSTASY

Deals with new developments in construction equipment. Several types of formwork, concrete mixers, batching and weighing equipment, concrete conveyers, and machines for the precasting industry are described and discussed in detail.

Flagstone for tilt-up panels

Contractors and Engineers, V. 57, No. 5, May 1960, pp. 10-12

The exterior tilt-up wall panels in this low-cost home for the aged were cast with a flagstone face on the exterior of the tilt-up slab. The erected panel looks like a masonry wall but is built at less than half the cost. In-

terior corridor walls were built of plain tilt-up slabs. Average size of panels was about 8 ft x 12 ft x 8 in. Interior panels averaged 10 x 12 ft with a 6 in. thickness.

After 1 in. of sand was spread inside the forms for the wall slabs, flagstone was pressed into place. A 1/2-in. layer of mortar held the flagstone in place. After placing two layers of 6 x 6–6/6 wire mesh on top of the mortar the structural concrete was placed.

Epoxy adhesives in concrete construction

R. W. Gaul and A. J. Apton, *Civil Engineering*, V. 29, No. 11, Nov. 1959, pp. 50-52

Discusses the uses, properties, preparations, and application of epoxy adhesives. Some of its uses are: bonding traffic buttons to pavements; repairing sidewalks and gutters; repair of chuck holes; bonding extruded concrete curb dividers onto existing pavement; and anchoring transverse dowels in runway slabs.

Dams

10,000,000 tons of aggregates

Harry F. Utley, *Pit and Quarry*, V. 52, No. 10, Apr. 1960, pp. 138-145, 148

A description of the aggregate plant, ready-mixed concrete plant, and pozzolan plant for Glen Canyon Dam in Arizona.

Thermic behavior of concrete dams (in Portuguese)

A. F. da Silveira, *Memorandum* No. 125, Laboratorio Nacional de Engenharia Civil (Lisbon), 1958, 31 pp.

Applied Mechanics Reviews
V. 13, No. 4, Apr. 1960

First part of paper discusses the influence of climatic factors, material properties, and construction methods upon the thermic behavior of concrete dams.

Second part presents a practical example of temperature prediction in a dam for the cases of natural cooling and artificial cooling by cold water flowing through pipes embedded in the concrete.

Design

Reinforced concrete column tables —Ultimate strength design

Hugh F. Fenlon, Hugh F. Fenlon, Atlanta, 1960, 316 pp., $15

This book presents reinforced concrete column tables which have been specially prepared to serve the needs of practicing structural engineers, architects, and designers. The tables are based on the ACI Building Code (ACI 318-56). The introduction explains the responsibilities of the column designer in selecting the size and reinforcement of columns and the materials. One section explains the basic formulas for rectangular columns with tension controlling, compression controlling, and reinforcement on tension and compression faces only. It also gives the basic formulas for round columns.

The sections include the usual concrete strengths shown in most reference material. The yield strengths for the reinforcement are for the common billet and rail steel bars. Column sizes were selected to include a wide range of rectangular columns under the 24 x 24-in. size. Three types of reinforcement were chosen for the tables in addition to the bars equally divided between the tension and compression faces. Bars were added on the tension face alone for the best use of steel under the low load, high moment conditions. They were added on the axis of bending to complement the face bars and to gain a more even distribution of bars in the columns. Finally, the symmetrical arrangements were included because of their simplicity and common usage. The ties or spirals for each column are shown on the individual tables and have been chosen for maximum economy.

Sixty-three column sizes are given in each of four sections for rectangular tied columns and 71 bar arrangements were chosen. Twelve column sizes are given in each of four sections for spirally reinforced columns with 44

arrangements of reinforcement. The bars were chosen to provide a gradual increase in the column values with an economical choice of bar sizes. Each column table contains the moment capacity for each column load of a load series covering the full range of allowable loads for the reinforcements shown. The load and moment values in the column tables are design values in the sense that the ultimate strength values have been reduced by the load factor and no load factor therefore need be applied to the live and dead loads. These combinations of column sizes, strengths of materials, and reinforcement arrangements, total 8568 different column designs which should be an invaluable tool.

Orthogonal gridworks loaded normally to their planes

IGNACIO MARTIN and JOSE HERNANDEZ, *Proceedings*, ASCE, V. 86, ST1, Jan. 1960, pp. 1-12
AUTHORS' SUMMARY

The torsional rigidity of the bars should be taken into consideration in this solution of a gridwork, since its behavior is similar to a two-way slab. The solution presented for this problem, which generally is highly indeterminate, allows the introduction of prestressing forces.

A set of simultaneous slope-deflection-gyration equations is established to be solved by electronic computer. A prestressed concrete deck, 177 x 177 ft, with beams spaced 10.8 ft apart in each direction, has been solved by this method.

Shell-structures in foundations (in Hungarian)

E. BOLCSKEI, *Melyepitestudomanyi Szemie* (Budapest), V. 9, No. 2, Feb. 1959, pp. 72-73
APPLIED MECHANICS REVIEWS V. 13, No. 4, Apr. 1960

Author claims a new field of application for shell construction and presents some invert shell forms which might well be applied in foundation design. The easy shaping of earth excavation makes the omission of extensive shoring and formwork possible and the saving in construction material is also well apparent. No suggestions for actual dimensioning or structural details are given; only the rough idea of application is shown for footings, strip foundations, and for rafts. The question of bending and of stress concentrations at the boundaries is not mentioned, although in this case — where the big concentrated loads transmitted by the columns must be spread on the soil by the invert shells—these must play a predominant role, and without deeper investigation the real economical value of the idea can't be judged.

Structural mechanics

SAMUEL T. CARPENTER, John Wiley and Sons, Inc., New York, 1960, 538 pp., $9.50

This text is primarily intended for use in undergraduate instruction in a course on indeterminate structures. It sets forth the connection between structural theory and engineering mechanics and presents the fundamental principles to provide a sound background for further study in the field of structures. Emphasis is placed on the mathematical treatment of structural theory bearing on indeterminate structures as well as the introduction of topics from advanced mechanics of materials. The author combines coverage of the traditional subject areas with material drawn from the advanced courses.

Design of slab type reinforced concrete stairways

A. C. LIEBENBERG, *The Structural Engineer* (London), V. 38, No. 5, May 1960, pp. 156-164
AUTHOR'S SUMMARY

The author develops a method of design of slab type stairways, based on deductions made from full scale and scale model tests, which incorporates the extensional stiffness produced by the interaction of the stair flights and landings and which has been successfully applied in practice.

Although the simple analytical procedure does not yield "exact" solu-

tions, the inaccuracies involved appear to be small for most cases occurring in practice. Not only does the method lead to economy in design but the architecture can in many cases be enhanced by the elimination of beams and columns without the use of excessively thick sections, which would be required if the design procedures used in normal practice were employed.

Two methods of calculation of paraboloid shells of revolution over a regular triangular base

P. Csonka, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 411-416
Author's Summary

Deals with the analysis of shells resting on edge arches, which can resist forces acting only in their planes. The methods are based on the well-known assumptions of the membrane theory, and on the assumption that the deformations of the shell and edge arches are independent of one another. Two methods of calculation are presented.

The first method can be applied only to the case of a circular symmetrical load system. To obtain the stress function, secondary functions which can be eliminated on the periphery of the base triangle are introduced. For each secondary function there is a circular symmetrical load system. The stress function is constructed as a linear combination of these secondary functions in such a way that the corresponding load system differs as little as possible from the actual one.

The second method can be applied to cases of any vertical loading. The calculations are carried out on a regular triangular grid. The values of the stress function are first determined on the inner points of the grid, after which the stress function is continued with the help of the differential equation beyond the periphery with regard to the actual boundary conditions.

The paper includes a numerical example proving the simplicity and practicability of both procedures.

Towards direct determination of the cross section of reinforced concrete members of rectangular section (Zur direkten Querschnittbestimmung von Stahlbetonrechteckquerschnitten)

H. H. Zerrenthin, *Der Bauingenieur* (Berlin), V. 34, No. 7, July 1959, pp. 273-276
Reviewed by Aron L. Mirsky

Develops and presents charts from which slab thickness h can be read off for various values of distributed load q [dead load (not including weight of slab itself) plus live load] and span length l, for six different combinations of two concretes and three reinforcing steels. Modification to permit application to rectangular slabs is indicated.

Contribution to the calculation of effective length of vertical legs of rigid frames with cranked and curved beams (Beitrag zur Knicklaengenberechnung lotrechter Rahmenstiele im Falle geknickter und bogenfoermiger Riegel)

J. Appeltauer and Th. Barta, *Der Bauingenieur* (Berlin), V. 34, No. 8, Aug. 1959, pp. 320-323
Reviewed by Aron L. Mirsky

Develops expressions for analysis of two-legged frames with two, zero, or three (symmetrical) hinges, with cross-members formed of one, two, three, or four straight pieces or curved (parabolic or circular), and loaded by concentrated loads applied at the knees collinearly with the axes of the legs. Extension to frames with multiple bays or multiple stories is indicated.

New method for calculating ferroconcrete structures of the type of slanting thin-walled spherical shells of variable thickness (in Russian)

B. S. Zharmagambetov, *Izvestiya Akademii Nauk Kazakhskoi SSR*, Seriya Gornogo Dela, Metallurgi, Str-va i Stroimaterialov, No. 3, 1957, pp. 3-12; *Referativnyi Zhurnal, Mekhanika* (Moscow), No. 1, 1959, Review 732
Applied Mechanics Reviews
V. 13, No. 5, May 1960

The boundary problem is investigated regarding slanting spherical shells of variable thickness, subjected to axially symmetrical deformation.

Note

Copies of books and articles reviewed are not available from ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In some cases ACI can furnish addresses of publications added later.

The initial equations appear to be Meissner's differential equations. It is accepted that the thickness of the shell changes proportionally to the distance of a point on the mean surface to the axis of symmetry; Poisson's coefficient is taken to be equal to zero; the solution is obtained in terms of Bessel functions. Approximate formulas are given for the deflection moments, for forces in the median surface and transpositions. Particular cases are examined of the loading of the shell along the edges by forces, the plane of which is perpendicular to the axis of symmetry or the moments.

Torsion or twisting: On the influence of twisting moments in plates with biaxial flexure (Torsion oder Drillung. Ueber den Einfluss der Drillmomente in Platten mit zweiachsiger Biegung)

J. HAHN, *Der Bauingenieur* (Berlin), V. 34, No. 8, Aug. 1959, pp. 296-299
Reviewed by ARON L. MIRSKY

Although analysis would seem to indicate high stresses due to biaxial flexure (dishing) in long rectangular slabs freely supported on either all four sides or on three sides (two short sides plus one long side), experience indicates most such slabs have performed satisfactorily even though not designed or reinforced for such shear plus flexural stresses, and that the requirements of German Standard DIN 1045 are thus overcautious. Author suggests designing for flexure based on the principal flexural moments, and reinforcing the slab corners and anchoring or weighting them against uplift. (For erratum, see *Der Bauingenieur*, Dec. 1959, p. 488)

Materials

Movement of sodium with water in neat portland cement

R. C. HALL and J. M. RHODES, *ASTM Bulletin*, No. 245, Apr. 1960, pp. 66-69
AUTHORS' SUMMARY

Movement of sodium in neat cement was studied by the aid of radioactive tracer techniques. Sodium was distributed through a slice having dimensions 1 x 1 x about 0.1 in. thick. This slice was subjected to high humidity on one side and low humidity on the other. Sodium was found to accumulate on the side of dry atmosphere. The slice was then placed in atmospheric air, and the surface concentrations tended to approach one another.

These are considered as evidence that: (1) sodium moves freely with water in cement, (2) a portion of the sodium tends to be bound (at least to some degree) with the cement and is thus immobilized, and (3) sodium tends to distribute itself with uniform concentration through a mass of concrete.

Physical and chemical properties of cement and aggregate in concrete

Bulletin No. 239, Highway Research Board, Feb. 1960, 65 pp., $1.20

The bulletin contains four related papers:

"Relationship of Pore-Size Distribution and Other Rock Properties to Serviceability of Some Carbonate Aggregates," by Carl L. Hiltrop and John Lemish, presents data whose development may ultimately lead to a description of a typical "poor" aggregate.

"Correlation Between Chemical and Mortar Bar Tests for Potential Alkali Reactivity of Concrete Aggregates," by Bernard Chaiken and W. J. Halstead, shows the necessity for use of both tests to characterize some aggregates.

"Silification of Carbonate Aggregates in Concrete," by Ramon E. Bisque and

John Lemish, shows more conclusive data on this theory than presented in a related paper in 1958 (*Bulletin* No. 196, Highway Research Board). Substantiating data are claimed to show that silica is introduced into some carbonate aggregate from the cement paste.

"Cement-Pozzolan Reactions," by E. J. Benton, reports on studies undertaken to develop information which will eventually lead to establishment of better criteria for pozzolan, mix proportions, and cement type to achieve optimum results.

Hydrophobic cements (in German)

FRIEDRICH-KARL SCHLUNZ, *Silikattechnik* (Leipzig), V. 10, No. 11, 1959, pp. 556-559
CERAMIC ABSTRACTS V. 43, No. 6, June 1960

Hydraulic binders lose much of their activity during storage, sometimes 30 percent or more in 6 months. This loss can be counteracted by organic surface-active additions to make the binders hydrophobic through an oriented adsorption by which the finest particles of the binder are enveloped by a film. Author describes experiments made with additions of 0.1 to 0.3 percent oleic or stearic acid and 0.15 to 0.5 percent producer tar. Portland cement and sulfate cement showed a remarkable improvement in mechanical strength after 3 months.

Shrinking aggregates in concrete

H. ROPER, *Special Technical Report* No. 502, South African Council for Scientific and Industrial Research, Pretoria, 1959, 136 pp.

Examination of structures in the Union of South Africa indicates that excessive shrinkage is a widespread problem. The coarse aggregates and sands derived from particular types of rocks (Karroo system) are responsible for large dimensional changes due to moisture content change in cement-bonded products containing them. The aggregates which must be regarded as of doubtful suitability for use in concrete construction, or for plasters and mortars, are the sedimentary rocks of the system, weathered basalts, weathered dolerites, and most of the sands derived from these rock types. Unweathered dolerite, an igneous rock often intrusive into these sediments, has been found to be satisfactory as a coarse aggregate.

Laboratory studies on the sands, rocks, and cement-bonded products indicated that the proportioning of the mix, the type and duration of curing, the period of observation, method of drying, and type of cement, all influenced the shrinkage of cement products. Laboratory tests revealed considerable variations dependent on the type of aggregate, the grading of the aggregate, and the water-cement ratio. Physical and chemical studies showed that the rocks which cause excessive dimensional change in concretes are themselves subject to dimensional change with changes in moisture content. Using statistical methods it was determined that the surface area of the aggregate is an important factor in determining the magnitude of shrinkage, and that the elasticity of the aggregate plays little part in shrinkage of mortar bars.

After evaluating several techniques for field test to identify aggregates which could lead to excessive dimensional instability in cement-bonded products, a portable dial gage comparator was devised. The recommended field test procedure using this instrument is given as an appendix.

The coming role of pozzolans. Part V—Product uniformity: key to successful pozzolan production

WOLF G. BAUER, *Pit and Quarry*, V. 52, No. 10, Apr. 1960, pp. 132-136, 148

Pertinent factors in applied process control are discussed as well as automation in grinding regulation. In view of the fact that pozzolans cannot be controlled on a chemical compound basis by preadjustment and raw material blending, pozzolan processing must be provided with exacting controls. It follows that deposit uniformity is a

primary prerequisite for successful pozzolan manufacture. [See also Current Reviews, ACI JOURNAL, V. 31 (*Proceedings* V. 56): No. 10, Apr. 1960, p. 1084; No. 11, May 1960, p. 1198.]

Admixtures . . . fifth ingredient for concrete

Consulting Engineer, V. 14, No. 4, Apr. 1960, pp. 93-102

Reviewed by ARON L. MIRSKY

While giving no technical data, this report summarizes the present status of admixtures as respectable elements, not for making bad concrete good, but for making good concrete better. They have moved "from the patent medicine shelf to the prescription drug department," and it is up to the engineer and specification writer to specify exactly what he wants to accomplish his purpost. A sample specification (by Parsons, Brinckerhoff, Quade, and Douglas) is appended.

Pavements

Concrete roads (in Dutch)

J. WEVER, *Ingenieur's Grav.*, V. 71, No. 21, 1959, pp. 63-75

ROAD ABSTRACTS Aug. 1959

The design of unreinforced concrete slabs used in Holland is considered, and mention is made of the design rules of G. Pickett. In Holland difficulties are experienced in the load transfer across expansion joints. Results of tests indicate that unreinforced concrete slabs with contraction and construction joints only, or prestressed concrete slabs without joints are to be recommended.

Highway engineering handbook

KENNETH B. WOODS (Editor-in-Chief), McGraw-Hill Book Co., Inc., New York, 1960, 1696 pp., $25

This handbook covers almost every aspect of highway engineering with the exception of structures. It is an authoritative compilation of engineering data covering the essential features of operations, design, and construction of highways with the subject matter presented in a clear concise and usable manner. The handbook is presented in four parts of seven sections each and was prepared by 47 leading authorities in the various fields. Part 1 covers material in the general field of administration, economics, planning, and traffic engineering. Part 2 covers the general subject of soils while Part 3 includes a concise and useful treatment on contracts and specifications while covering all of the basic highway materials. Part 4 is dedicated to highway design, construction, maintenance, and landscaping.

An attempt has been made to provide the practicing highway engineer, the consultant, the student, the teacher, and even the researcher, with a highway engineering handbook on planning, traffic, administration, finance, economics, materials, design of mixtures, soil mechanics, design, construction, and maintenance of highways, that is an authoritative and concise reference volume with much new material previously unavailable in highway literature and with the other material presented in a brief up-to-date and interesting fashion.

Highway engineering

L. J. RITTER, JR. and R. J. PAQUETTE, 2nd Edition, Ronald Press Co., New York, 1960, 751 pp., $10

This second edition has been carefully revised to bring the book in line with recent developments in the field of highway engineering. In some chapters major revisions have been necessary because of technological changes and advances. The arrangement of material corresponds to the order in which factors in the development of a highway or highway system commonly occur. Good coverage is given to the more general aspects of highway engineering—administration, economics and finance, and planning before going into the general considerations of geometric design, soil engineering, drainage, surveys and plans, documents, supervision, and earthwork

operations. The various types of roads and construction materials are then covered under bituminous materials, design of flexible pavements, soil stabilization, macadam roads, bituminous surface treatments, bituminous plant-mix surfaces, high-type bituminous pavements, design and construction of portland cement concrete pavements, and finally highway maintenance.

This book is designed both for the use of the highway engineer as a reference and as a basic textbook for civil engineering students.

Building the London - Birmingham motorway

W. KIRBY LAING, *Engineering* (London), V. 188, No. 4873, Sept. 11, 1959, pp. 175-179
Reviewed by ARON L. MIRSKY

Covers briefly planning, organization, paving, bridges, and excavation. Should furnish some interesting comparisons for American planners and engineers.*

*See also *The Engineer*, Nov. 6 and 13, 1959, pp. 544-548 and 586-591; Current Reviews, ACI JOURNAL, Aug. 1960, p. 237.

Experimental continuously reinforced concrete pavements: Progress reports — 1959

Bulletin No. 238, Highway Research Board, Feb. 1960, 104 pp., $2

Continuously reinforced concrete pavements in Pennsylvania

F. C. WITKOSKI and R. K. SHAFFER, pp. 1-22 (including discussion)

The Hamburg project in Berks County has attained 1 year of service performance. The York project is now 2 years old, although it, too, has been opened to traffic only 1 year. This report describes the general condition of each project and presents the combined results of a comprehensive crack frequency and width survey on both pavements. A study is made of the different crack patterns evolved as a result of paving in opposite seasons of the year. Further analysis is made of the effect of various pavement thicknesses upon service performance of the Hamburg roadway and of the effect of various depths of subbase material.

A scattered number of detrimental transverse cracks appeared on the Hamburg project within several months after completion. The nature of these cracks and the investigation to determine their cause is described, together with an account of the subsequent repair of these damaged areas. The specifications for these repairs are outlined, emphasizing the necessary precautions required for this work. A detailed description is given of the methods employed in pavement removal, in restoring the continuity of the steel, and in replacing the damaged concrete.

Observations on the behavior of continuously reinforced concrete pavements in Pennsylvania

I. J. TAYLOR and W. J. ENEY, pp. 23-38 (including discussion)

Summarizes the results of 3 years of sponsored research on continuously reinforced concrete pavements as conducted by the Fritz Engineering Laboratory of Lehigh University.

Test data and conclusions based on instrumentation, physical measurements, and observations of two current pavement projects are reviewed, and a general pattern of behavior for continuous pavements is established.

Some of the weaknesses found in existing pavements are described and given consideration in suggestions for design and construction improvements in continuous pavements.

Laboratory research on pavements continuously reinforced with welded wire fabric

M. J. GUTZWILLER and J. L. WALING, pp. 39-47

Research has been conducted at the Structural Engineering Laboratory of the School of Civil Engineering, Purdue University, on reinforced concrete pavements using welded wire fabric as the principal reinforcement. The techniques as used in the laboratory are presented.

The specimens chosen were 28 ft long by 3 ft wide by 8 in. thick. The reinforcement consisted of either 6 x 12 0/3; 6 x 12, 00000/0; or 4 x 12, 00000/0 welded wire fabric. The specimens were cast in a portable form in

which the amount of steel, the location of the steel, and the depth of slab could be varied. Each of the specimens was fabricated with weakened planes such that the slabs would crack at definite locations in the testing region of the slab. This permitted measurement of strains in the fabric at these predetermined cracks.

The slabs were tested on an elastic subgrade having a subgrade modulus of approximately 160 pci, although the subgrade modulus could be varied within reasonable limits. The slab specimens were loaded with vertical static loads to simulate traffic loads and horizontal loads to simulate stresses induced by temperature changes. Electric SR-4 strain gages were placed at various locations on the fabric to determine the stresses in the fabric. Vertical deflections of the slabs were obtained by use of Federal dial indicators, and crack widths or surface strains in the concrete were obtained by use of a Whittemore strain gage.

Stresses and deflections in concrete pavements continuously reinforced with welded wire fabric

M. J. Gutzwiller and J. L. Waling, pp. 48-63

Laboratory experiments on simulated continuously reinforced concrete slabs are summarized, with experimental results pertaining to slab deflections, crack widths, and stresses in welded wire fabric reinforcement receiving most emphasis. Some of the significant findings of these laboratory experiments are compared with field observations reported in the literature and several criteria are suggested for optimum structural design of continuously reinforced pavements.

Crack formation in continuously reinforced pavements

M. J. Gutzwiller and J. L. Waling, pp. 64-77

The mechanics of crack formation in continuously reinforced pavement slabs, as understood from the literature on several field test pavements, are summarized.

Results of a series of laboratory experiments on simulated continuously reinforced concrete slabs are given, with those results pertaining to the formation of cracks being emphasized. The findings of these laboratory experiments are correlated with the field observations reported in the literature and one criterion for the design of continuously reinforced pavements is suggested.

Bond and transflexural anchorage behavior of welded wire fabric

William Zuk, pp. 78-93 (including discussion)

Mathematical theories are developed to predict the interactive bond-anchorage behavior of welded wire fabric when subject to a pull-out action, simulating the behavior of welded wire fabric in continuously reinforced concrete pavements. The theories are predicted on the concepts that the bond between the smooth longitudinal wires and the concrete is a function of minute movement between the steel and the concrete and that the anchorage of the transverse wires is achieved through the restrained flexural action of the transverse wires.

A summary of various general experimental techniques that other investigators have developed to measure bond is also presented as a background to the author's method, not hitherto used on small bars, in which electrical resistance strain gages are embedded in slots inside the bar. Based on this method, a description of the experiments on bond-anchorage behavior of welded wire fabric in concrete is presented. The tests are of a pull-out nature encompassing bond-anchorage behavior at first loading and after repeated cyclic loading.

A comparison of the theory and experiments shows general agreement.

Mechanics of continuously reinforced concrete pavements

M. M. Miller, Jr. and M. J. Gutzwiller, pp. 94-104

This investigation is primarily concerned with the deflections and the resulting stresses in a continuously re-

inforced concrete pavement, loaded simultaneously with longitudinal and transverse loads. The theory developed for the computation of the deflections is based on the common assumptions used in the theory of continuous beams on elastic foundations, as well as three assumptions concerned with a cracked slab on an inelastic foundation.

The main assumptions in the first class are: (a) The deflection at a point some distance from the transverse load is zero; and (b) the subgrade modulus is constant throughout the full range of deflection. The assumptions peculiar to this problem are: (a) The cracks formed by volume changes in the pavement are equally spaced; (b) the segments between cracks are assumed to be straight; and (c) the moment at a crack is some function of the angle change (that is, $M/\phi = C$).

By considering the geometry of the deflected pavement and the equilibrium of the individual segments, a series of simultaneous equations may be written in terms of either the deflections or the angle changes at the cracks. Equations for shear and moment at the cracks are easily written.

The equations are in such a form as to consider any combination of pavement length, crack spacing, subgrade modulus, transverse load, and longitudinal load. In the solutions presented here, M/ϕ is considered equal to a constant, C. Experimentation is in progress to evaluate C for a series of pavement types.

A new way to pretension prestressed pavement

LEV ZETLIN, *Consulting Engineer*, V. 14, No. 2, Feb. 1960, pp. 80-85

Reviewed by ARON L. MIRSKY

Utilizing the fact that tensioning one "spoke" of a wheel (which may be elliptical or oblong rather than circular) puts the other spokes into tension and the rim into compression, author advocates using large-dimension rims as temporary (portable) abutments for the construction of large-expanse pavements by pretensioning. Auxiliary tension cables are used to stabilize the rim. Article gives theory of method, including a discussion of the merits of two-way *versus* one-way pretensioning and of pretensioning *versus* post-tensioning, and discusses such refinements as providing continuity between sections. The design of a jet taxiway for construction by this method is described briefly as an example.

Recommended associated reading: "Radistres—a New Idea in Prestressed Panels," *Concrete*, V. 68, No. 1, Jan. 1960, pp. 16-19.

Prestressed Concrete

Proceedings of Third Congress of the Federation Internationale de la Precontrainte—Discussion

Fédération Internationale de la Précontrainte, 1959, 210 pp., $40 [price includes two volumes, one containing the papers (766 pp.), the other the discussion. Published for FIP by Cement and Concrete Association, London]

Contains discussion of papers presented at the Third Congress of the Fédération Internationale de la Précontrainte, Berlin, May 5-10, 1958. [For a review of the volume containing the technical papers see ACI JOURNAL, V. 31, No. 2, Aug. 1959 (*Proceedings* V. 56), p. 183.]

Redundant construction of prestressed ferroconcrete (in Russian)

B. F. GORYUNOV, Gos Izdvo Lit. po Str-vu i Arkhitekture, Leningrad-Moscow, 1957, 207 pp. 7r., 10k; *Referativnyi Zhurnal, Mekhanika* (Moscow), No. 10, 1958, Review 11802

APPLIED MECHANICS REVIEWS V. 13, No. 5, May 1960

The basic state of affairs in connection with the designing and calculations of prestressed reinforced redundant constructions (in concrete) is surveyed, covering continuous beams, frames, plates, and shells. The most attention is devoted to continuous beams. Based on an analysis of the results of experiments by Magnel, Guyon, and Lin, recommendations are made that: the deflection moments along the length of the beam up to the appearance of cracks be calculated by

the elastic stage, and the moments producing the cracks—taking into account the plastic deformations, which limit the loading—by taking into account the complete redistribution of the moments along the length of the beam; but in all cases, where today there is not in existence a worked-out scheme for the consideration of the redistribution of moments, it is permissible to determine the magnitude of the deflection moments by the elastic stage. These conclusions also apply to other types of redundant constructions made with prestressed ferroconcrete. Formulas are advanced and examples given for the calculations of redundant beams of prestressed ferroconcrete by the elastic stage, using the method of forces, the method of angular deformations, and the combined method. To evaluate the magnitudes of the forces, which are developed by shrinkage and creep, the results of the calculations for a double-span beam are furnished. For the calculations for the boundary state Gvozdev's method is recommended. The different methods of calculation for frames by the elastic stage are investigated using an example of a two-hinged frame. On this example also is given an evaluation of the influence of shrinkage and creep, and an investigation carried out on the basic situation regarding the calculations for frames in accordance with their carrying capacity. Methods of calculation for rectangular plates by the elastic stages and numerical examples are given for cases of support on two, three, and four sides. Resulting from the calculations made a recommendation is put forward to distribute the prestressed reinforcement between two mutually perpendicular directions in proportion to the moments due to external loading. A round plate is examined. Examples are provided of prestressed reinforced shells, of some concepts on the calculation of shells as bodies of rotation, and calculations for cylindrical and spherical shells. At the beginning of the book general information is to be found and at the end, evaluations of the advantages accruing from the adoption of redundant constructions made of normal prestressed ferroconcrete as against similar constructions made of ordinary ferroconcrete (with 15-20 percent economy) and redundant constructions made of prestressed ferroconcrete (with a decrease of quantity of reinforcement of 20-30 percent).

Prestressed tapered joists for roofs

MILO S. KETCHUM, *Journal*, Prestressed Concrete Institute, V. 5, No. 1, Mar. 1960, pp. 25-28

AUTHOR'S SUMMARY

Maximum economy of material in prestressed concrete roof construction indicates that the joists be sloped to a maximum depth at the peak with an absolute minimum depth at the supports. Using this criterion, a series of single tee joists were developed for use with modular forms. The area at the support is increased by increasing the width of the flange to the full width of the joist. Some of the problems of design are discussed.

Theory of anchorage of prestressed reinforcement in concrete (in Hungarian)

G. TASSI, *Epites- es Kozlekedestudomanyi Kozlemenyek* (Budapest), No. 1-2, 1959, pp. 217-258

APPLIED MECHANICS REVIEWS V. 13, No. 5, May 1960

The distribution of bond forces arising between steel and concrete is investigated. Starting from functions which are in good agreement with experience, author develops a general theory. From this, formulas are obtained, suitable for computing the bond forces in question. The theories of other authors are also recapitulated and criticized.

Folded plate concrete roof is post-tensioned in place

JAMES OWEN POWER, *Journal*, Prestressed Concrete Institute, V. 5, No. 1, Mar. 1960, pp. 29-34

The design problem for a bowling alley required provision of a 34,000

sq ft area unobstructed by columns. The structural system was required to be economical and attractive in appearance and construction time was limited. The solution consisted of a corrugated concrete slab, spanning approximately 120 ft, and extending transversely 286 ft for the length of the building. The over-all depth is 6 ft; the horizontal module is 22 ft. Top and bottom flanges are 8 in. thick and the inclined webs are 5 in.

This folded slab is supported at one end of the span on concrete columns which support the center of the bottom flange. Precast concrete frames cast to the shape of the corrugation were used as window grilles as well as diaphragms. At the other end, the roof spans to a continuous 12 in. block wall framed with concrete columns and beams.

Each interior corrugation is reinforced with 14 prestressing cables; eight of these are placed straight in the bottom flange and three are placed parabolically in each web. Mild steel was used to provide for temperature stresses and load distribution in the transverse direction. Special mild steel reinforcement was added in the top corrugation to provide for construction loads.

Design of prestressed concrete beams by computer

Joseph J. Bonasia, *Proceedings*, ASCE, V. 86, ST 4, Apr. 1960, pp. 129-155

Author's Summary

Illustrates how an electronic digital computer is advantageously used to select and design standard post-tensioned prestressed concrete beams. Two charts are presented which provide information for the rapid selection of satisfactory beams used as interior stringers in simple span, composite beam highway bridges with AASHO H20-44 live load. A description of the method of obtaining the required prestressing force and its location for the results obtained is also presented.

Corrosion tests on prestressed concrete wire and strand

Howard J. Godfrey, *Journal*, Prestressed Concrete Institute, V. 5, No. 1, Mar. 1960, pp. 45-51

Illustrates some of the conditions which should be avoided to prevent corrosion, both before and after the steel has been placed in the concrete. Experiments with hard drawn stress relieved wire and oil tempered wire are reported. The tests indicated that oil tempered wire fails more readily under stress-corrosion conditions than does the hard drawn stress relieved wire.

The results of a study on the effect of calcium chloride on the corrosion of steel wire and strand are also described. The tests indicate that calcium chloride should not be permitted in prestressed concrete structures.

Bundled strand: Its effect on the performance of prestressed beams

Ross H. Bryan *Journal*, Prestressed Concrete Institute, V. 5, No. 1, Mar. 1960, pp. 73-85

Author's Summary

Describes the results obtained from testing 24 beams with bundled and spaced strands. The prestressing force was maintained constant and d was varied in 4 x 11 1/2-in. specimens. Four test series were made, having four values of d, with three spaced strand specimens and three bundled strand specimens made for each series. The resulting influence on ultimate strength, cracking load, crack pattern, and deflection is tabulated and discussed. The test and computed ultimate strength loads of the beams are compared, using several current methods of computing ultimate strength.

Prestressed concrete dock gate

The Engineer (London), V. 209, No. 5424, Jan. 8, 1960, p. 72

Reviewed by Aron L. Mirsky

Brief description, with sketches, of another application of prestressed concrete. Floating dock gate is 62.6 m long over-all, has a 13.6 m beam and a height of 15.01 m (keel to gangway),

and weighs 4500 tonnes (1 tonne = 0.9842 ton) empty; 2066 tonnes of water are admitted to the interior compartments to sink the gate and apply the necessary force to the sill. The center compartment of the 15 into which the interior is divided contains the pumping machinery. The gate forms part of the largest drydock on the European continent, the "Elbe 17," at Hamburg-Steinwerder, which has a length of 350 m, a width of 60 m, and a depth of 9 m at mean high tide.

Papers presented at the Third General Reunion of the Spanish Association of Prestressed Concrete (III Asamblea General de la Asociacion Espanola del Hormigon Pretensado)

Bulletin No. 189, 190, and 191, Instituto Técnico de la Construcción y del Cemento (Madrid), 1958, 47, 39, and 52 pp.
Reviewed by ROGER DIAZ DE COSSIO

Papers deal with practical applications of prestressed concrete in Spain. All well-known Spanish engineers are represented, among them Torroja, Paez, Fernandez Casado, etc. An interesting paper by Torroja describes a prestressed structure without prestressing steel. It is a water tank with two concentric walls. The outside wall is ordinary reinforced concrete and the inner wall is made of solid bricks. The space between the walls is injected with cement and gravel grout that, when hardened, puts the inner wall into compression. Other tanks of this general type are described.

Concrete-tie service tests begin

MERWIN DICK, *Railway Age*, V. 148, No. 11, Mar. 14, 1960, pp. 9-10, 15-17
Reviewed by ARON L. MIRSKY

Interesting story reporting completion of a test installation of prestressed concrete ties on the Seaboard Air Line near Tampa, Fla. Installation consists of 600 ties on quarter-mile length of 115 lb continuous (welded) rail of single-track main line. Similar installation on Atlantic Coast Line near Four Oaks, N. C. was in progress at the time. Basic concepts governing design of ties being tested, costs, etc., are also noted. [cf., *Railway Age*, V. 147, No. 19, Nov. 9, 1959, pp. 20-21; "Current Reviews," ACI JOURNAL, V. 31, No. 10, Apr. 1960 (*Proceedings* V. 56), p. 1073]*

*See also "Railroads test concrete crossties," *Engineering News-Record*, V. 164, No. 10, Mar. 10, 1960, pp. 26-27

Properties of Concrete

Statistical aspects of fracture in concrete—I

MOTOO HORI, *Journal*, Physical Society of Japan (Tokyo), V. 14, Oct. 1959, pp. 1444-1452
BATTELLE TECHNICAL REVIEW Mar. 1960

An application of the theory of stochastic processes to the static fracture of nonhomogeneous brittle materials such as portland cement concrete is presented. It is concluded that the scatter observed in the breaking time or the static strength of concrete appears to be a result of inherent characteristics of the material itself.

Note on the maturity and creep of concrete

A. D. Ross, *RILEM Bulletin* (Paris), No. 1, New Series, Mar. 1959, pp. 55-57
From AUTHOR'S SUMMARY

From studies carried out in numerous laboratories we know that creep in concrete, under constant load, is a function of age at which the load is applied. The temperature of curing before loading also has a marked influence on creep in concrete. The increase in the strength of the concrete is connected with its hardening, which depends on age and temperature of curing.

The temperature is measured in relation to a reference temperature thus defined: "At this temperature the chemical reactions diminish to the point where the strength does not increase." The hardening unit is thus expressed in deg C $\times$ hr and if we take —10 deg C as reference temperature we find that concretes have the same strength if they are at the same

temperature whether the latter be reached after a long period at low temperature or a short period at higher temperature.

Creep is also a function of the hardening at the time of first loading and it can be said that it is more sensitive to changes in hardening then the strength or the static modulus.

These results do not contradict previous experiments since the temperature is constant, the hardening is directly proportional to time and thus different creep values correspond to different loading ages.

Influence of dimensions and stress on the shrinkage and creep of concrete (in French)

G. Weil, *RILEM Bulletin* (Paris), No. 3, New Series, July 1959, p. 4

Author's Summary

The research described relates to the influence of dimensions, and the influence of the magnitude of the stress, on the shrinkage and creep of concrete. The first series of tests was carried out on cylindrical test pieces having diameters of 10, 20, 30, and 60 cm, with lengths of 40, 80, 120, and 180 cm. The second series of tests was carried out on cylindrical test pieces having a diameter of 10 cm and a length of 40 cm, to which were applied, from the 28th day, loads of 25, 45, 60, and 70 percent of their rupture strength.

The author gives the characteristics of the cements and aggregates, describes the preparation of the concrete, of the test pieces, the two methods of loading, the device for measuring shortenings, and gives the characteristics of the concretes. The strengths of the concrete cubes, 10 and 20 cm, and cylinders, 15 cm in diameter and 30 cm long, are given in relation to age in the two series of tests. The moduli of elasticity are measured on the various test pieces by ten loading-unloading cycles between 0 and 10 kg per sq cm in relation to time in the first series; they are given in relation to time and to maximum stress in the second series.

The results are shown by curves which give, in the first series, the influence of dimensions on shrinkage, on shrinkage and creep, and finally on creep alone (slowing up of phenomena). The author characterizes the creep by the following ratio:

$$\phi = \frac{\text{shortening due to creep}}{\text{elastic shortening under load}}$$

and gives the variations in this relationship with time for the different test pieces.

In the second series of tests the results are given for the same phenomena: shrinkage, shrinkage and creep, creep, in relation to time for the magnitude of stress planned. The test pieces have the same dimensions. The author finally gives the variations, measured at 270 days, of the creep with the stress and the variations of ϕ in time for a value of stress of 45 percent of the strength.

Tests with crystallization nuclei in concrete production to obtain high initial strengths (in German)

Helmut Schunack, *Silikattechnik* (Leipzig), V. 10, No. 7, 1959, pp. 326-330

Ceramic Abstracts V. 43, No. 6, June 1960

The setting time of concrete can be shortened by the addition of hardened finely ground cement (crystallization nuclei). If $CaCl_2$ or NaOH with sodium methyl siliconate is added in small amounts to the fresh cement, concrete of high early strength can be obtained. The optimum addition is about 2 percent (weight). Normally, concrete has 60 percent of the 28-day compressive strength after 3 days, but with the standard nuclei and with the chemical additions, the 3-day strengths were 80 and 100 percent or more, respectively. The 28-day strength showed a 50 percent increase over that of the standard sample. Flexural strengths behaved similarly. The crystallization nuclei are powdery and can be combined with other concrete additions as plasticizers and air-entraining agents, but they give no frost protection. The best

additions for portland cements were found to be nuclei with 0.5 percent each of sodium methyl siliconate and $CaCl_2$ and for blast-furnace cements 0.5 percent each of the siliconate and NaOH. The addition of 2 percent (weight) can be subtracted from the required amount of cement. Test results are shown in detail.

Hydraulic telemeter method for mechanical stresses and pressures (Hydraulisches Fernmessverfahren fur mechanische Spannungen und Drucke)

F. GLOTZL, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 4, Apr. 1959, pp. 111-113

Reviewed by FERDINAND S. ROSTASY

The author discusses the various sources of volume change in concrete and establishes the requirements a load cell has to meet. The load cell, developed by him, is a flat (⅛ in.) two-layer membrane with negligible bending stiffness. The acting stress is measured by oil pressure directly, whereas there is no limit to the distance between the gage and the indicator. Zero drift does not take place.

Some aspects of the strength of concrete

A. M. NEVILLE, *Civil Engineering and Public Works Review* (London), V. 54: No. 639, Oct. 1959, pp. 1153-1156; No. 640, Nov. 1959, pp. 1308-1310; No. 641, Dec. 1959, pp. 1435-1439

A theoretical discussion of the factors affecting the strength of concrete test specimens. Author finds strength to be a function of an underlying statistical distribution of strength of elements. According to this, strength and standard deviation of specimens depend on their size. General trends of strength behavior are found to be demonstrated experimentally to apply to various types of test specimens and systems of applied stress. It is further suggested that uniaxial compression and a biaxial compression-tension, as well as direct tension and flexure-tension, may produce conditions under which fracture takes place.

General

The architect has prime responsibility

E. D. JEFFERISS MATHEWS, *Engineering* (London), V. 188, No. 4884, Nov. 27, 1959, pp. 543-544

Reviewed by ARON L. MIRSKY

Author's thesis here is interesting both in its own right, and in comparison with his previous paper.* There the emphasis was all on collaboration; here the emphasis is on collaboration, but with the architect as leader of the team. It is he who is best suited, in all respects, to "design" buildings (as distinguished from civil engineering projects) for economy, efficiency, and aesthetics; but the engineer, while occupying a subordinate position as "servant collaborator," must be a true collaborator — he must collaborate, not merely obey. And, once the status of the contractor has been raised, once he is no longer held in public distrust, design should be a collaborative project of a team of architect, engineer, and contractor,† with the architect as leader. In the meantime, much improvement can be obtained via some form of combined technical education.

Reviewer would add that not only is this suggestion by author interesting in view of the current ASCE and NSPE activity regarding competitive bidding for engineering services, but that certain American owners (notably the insurance companies) are already using "consulting contractors" on their design teams.

In replying to a discusser (*Engineering*, Dec. 11, 1959, p. 599) the author (Jan. 1, 1960, p. 4) restates his main point: engineers are not trained in the appreciation of design in its broadest sense, hence in general are not qualified to lead the team. Reviewer, mindful of the amount of ivy being planted and carefully nurtured, would beg to differ.

Proceedings, Institution of Civil Engineers, V. 13, Aug. 1959, pp. 499-502; Current Reviews, ACI JOURNAL, V. 31, No. 12, June 1960 (*Proceedings* V. 56), p. 1316.

†He, like the architect and the engineer, to be appointed at the inception of the project, and not selected on the basis of competitive bidding—after the design is completed!

ACI NEWS LETTER

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Southwest to Tucson

13th Regional Meeting

THE ACI 13TH REGIONAL MEETING will be held in Tucson, the "sunshine city" of the Southwest, October 31 to November 2, with headquarters at the Pioneer Hotel.

The local planning committee has arranged a top-notch 3-day program, highlighted by joint ACI-ASTM technical sessions, scenic sightseeing tours along the Old Spanish Trail, and special activities for the ladies.

On the business side, Monday, October 31, will be devoted to technical committee meetings. The program on Tuesday, November 1, will include a morning session on construction and an afternoon session on research in design.

Wednesday, November 2, will feature concurrent technical sessions in the morning: an ACI design session and an ASTM research session. The sessions will be significantly different with the accent on papers and discussions of interest to both ACI and ASTM members.

A joint ACI-ASTM luncheon on the day of the concurrent sessions has been arranged with A. Allan Bates, vice-president of research and development, Portland Cement Association, Chicago, and currently president of ASTM, as guest speaker. Long active in ACI affairs, Dr. Bates is a member of the Institute Board of Direction.

The cosmopolitan city that is Tucson today offers plenty of local color, history, and

sightseeing ventures. Molded in the roughest traditions of the frontier, the dusty streets have broadened into modern, paved thoroughfares, but its basic charm and unique beauty still prevail.

On Tuesday, November 1, the local committee has planned a chartered bus trip to the "Old Tucson" movie set and the Sonora Southwest Desert Museum with a Western chuck wagon-style dinner later in the evening. The following day a bus trip into Mexico, luncheon at the Caverns dining room in Nogales, Sonora, and a shopping spree for the wives "south of the border" will keep the ladies well entertained.

The technical program for the meeting is as follows:

TECHNICAL PROGRAM

TUESDAY MORNING, NOV. 1

Construction Session

Missile Base Construction Problems

Lt. Col. G. H. Andrews, Strategic Air Command, U. S. Air Force, Offutt AFB, Omaha, Neb.

Timber Falsework Design and Usage

George Birkemeier, Timber Structures, Inc., Portland, Ore.

Effects of Recent Mexican Earthquakes on Reinforced Concrete Structures

R. D. de Cossio and Emilio Rosenblueth, University of Mexico, University City, Mexico D.F., Mexico

World's Largest Lift Slab, The Motorola Building, Phoenix

Charles Magadini, consulting engineer, Phoenix, Ariz.

TUESDAY AFTERNOON, NOV. 1

Research in Design Session

Solution of Structural Problems through Model Analysis

Jack R. Janney, Engineers Collaborative, Chicago

Full Scale Lightweight Experimental Bridge

Truman R. Jones and T. J. Hirsch, Texas Agricultural and Mechanical College, College Station

Failure of Small Reinforced Concrete Beams under Repeated Loads

J. R. Verna and T. C. Stelson, Carnegie Institute of Technology, Pittsburgh

Fatigue Properties of Lightweight Concrete

J. F. McLaughlin, Purdue University, Lafayette, Ind.

CONCURRENT TECHNICAL SESSIONS

WEDNESDAY MORNING, NOV. 2

ASTM Research Session

Water Vapor Permeability of Concrete

D. F. Griffin, Naval Civil Engineering Laboratory, Port Hueneme, Calif.

Effect of Steam Curing on Important Concrete Properties

Elmo C. Higginson, Bureau of Reclamation, Denver

Concrete Retempering Studies

M. J. Hawkins, Bureau of Reclamation, Ephrata, Wash.

Properties of an Expansive Cement for Chemical Prestress

Alexander Klein, T. Karby, and Milos Polivka, University of California, Berkeley

ACI Design Session

Prestressed Composite Design in Multistory Building Construction

Harry Edwards, Leap Associates, Lakeland, Fla.

Prestressed Girder Design, Testing, and Construction for Norton Building, Seattle

Arthur R. Anderson, Anderson, Birkeland, and Anderson, Tacoma

Hyperbolic Reinforced Concrete Cooling Towers

Paul Rogers, Paul Rogers and Associates, Chicago

Prismatic Folded Plates — A Design Procedure

A. A. Brielmaier, Washington University, St. Louis

WEDNESDAY NOON, NOV. 2

Joint ACI-ASTM Luncheon

Guest speaker — A. Allan Bates, Portland Cement Association, Chicago

WEDNESDAY AFTERNOON, NOV. 2
Joint ACI-ASTM Research Session

Heat Resistant Concrete for Missile Bases
Herman G. Protze, materials technologist, Boston

Abnormally Slow-hardening Concrete
L. H. Tuthill, R. F. Adams, S. N. Bailey, and R. W. Smith, California Department of Water Resources, Sacramento

The Use of Epoxy Resins on Highways
R. E. Brown, Shell Chemical Co., New York

Epoxy Resins as Structural Adhesives
J. D. Kriegh and T. C. Ewbank, University of Arizona, Tucson

Local committee

Gene M. Nordby, head, civil engineering department, University of Arizona, is general chairman of the meeting with Andrew Ross, vice-chairman and Joseph Toth, treasurer, both of the University of Arizona. Edward Mangotich, Portland Cement Association, is on the executive planning committee.

Others assisting in arranging the meeting are: Billy J. Shell, San Xavier Rock and Sand Co., technical program chairman; Lee Lawrence, civil engineer, finance chairman; Edward Mangotich, PCA, in charge of exhibits; Gene Anderson, civil engineer, publicity chairman; and Andrew Marum, consulting engineer, membership chairman.

Maurice LeGaard is in charge of the ladies' program; James Kriegh, educational contacts; and K. K. Kienow, registration. The latter three members of the local planning committee are all affiliated with the University of Arizona.

Solite promotes McLeroy

James H. McLeroy, Jr., has been appointed manager of the Solite Silica Division of Solite Corp., Richmand, Va. Mr. McLeroy joined Solite in 1959.

Southern California Chapter announces meeting schedule

Officers and committee chairmen of the ACI Southern California Chapter are planning an active season in chapter affairs, with the first fall dinner scheduled for Thursday, September 15.

Present plans call for chapter meetings on the third Thursday of alternate months: Nov. 17, 1960, and January 19, March 16, and May 18, 1961. The March date is that of the annual chapter meeting.

The September meeting will feature a program outlining problems and practice in hot weather concreting, finishing methods, and distinctive finishes. Members will present the program.

Preliminary plans for a later date contemplate a joint meeting with ASTM. Robert E. Tobin, Portland Cement Association, Los Angeles, heads the program committee in charge of arrangements for meeting features.

J. L. Peterson, J. L. Peterson, Inc., Long Beach, is chairman of the chapter's technical activities committee. Mr. Peterson is organizing a group of subcommittees to study building code problems and participate in local activity in that field, to study developments in such areas as fire ratings, precast, prestressed, and thin shell concrete, and to consider concrete problems which require attention and on which a public service may be rendered.

John L. Goetz, Southwestern Portland Cement Co., Los Angeles, as chairman of the membership committee, is organizing a committee to get the chapter message before present ACI members and prospective members by means of group mailings and individual effort, to ensure a steady and healthy growth of the chapter.

The finance committee, under chairman John W. McNerney, PCA, Los Angeles, and the committee on nominations, under chairman Harold Omsted, Los Angeles City Board of Education, are also actively carrying out their assignments.

Lightweight Concrete for Office Building

■ THE ATTRACTIVE Number One Davis Medical Office Building on Davis Island in Tampa Bay, Tampa, Fla., made wide use of lightweight concrete in the structural frame and interior and exterior precast elements. The building is of reinforced lightweight structural concrete of column and pan floor design. Interior partitions used expanded shale concrete masonry. Exterior wall panels were precast with an exposed decorative aggregate surface backed up by expanded shale concrete. The sun screens were precast concrete and the entrance canopy was of lightweight concrete.

The low masses around the first floor areas are lightweight concrete block faced with black ceramic tile. Exterior walls from second through seventh floors are in part glass and aluminum curtain walls with blue porcelain spandrel panels while other areas used precast concrete panels of lightweight aggregate surfaced with river gravel. Curtain walls on the east and west facades are shielded from the sun with white, precast concrete sunscreen. Concrete cantilevers serve as a sunshade on the north and south, support the sunscreens on the east and west, and facilitate window washing in both cases.

The structural frame is of lightweight Galite concrete, dry weight 100 lb per cu ft. Essentially, the floor framing is of pan construction, with 26 x 26-ft bays. Flat bands were haunched at the columns so as to keep the structural depth and the depth required for air conditioning ducts to a minimum. The building is made up of nine equal bays, the center bay housing the stair and elevator cores.

Each column is a "wet column" and the flat band and haunches were pierced to permit the supply and return water lines to pass through them. The building height, because of the thickness of the ceiling to floor construction, is an 80 x 80-ft cube from the ground to the top of the roof slab. A basement, the full area of the building, holds all the mechanical equipment. Because the building is on an island and one-half block from the bay and river, it had to be designed for approximately 6 ft of hydrostatic pressure. Because of the lightness of the structure, the building sits on 15-ton untreated wood piling.

The entrance canopy is supported on four 8 x 8-in. columns 8 ft on centers and cantilevers 25 ft in one direction. It was designed making use of folded

One of the unique features of the building is this unusual canopy at the east entrance. It is supported on four 8 x 8-in. columns 8 ft on centers and cover over the driveway is provided by a 25 ft cantilever section. It was built of lightweight concrete

Photos courtesy Georgia Lightweight Aggregate Co.

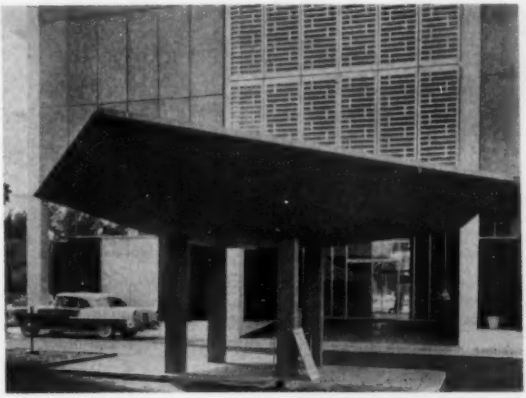


plate principles, using lightweight concrete.

All the permanent partitions and walls are 6-in. or 4-in. expanded shale concrete block. Again lightness was essential to keep down the dead load and to make the concept of the building possible and economical.

The first floor, or entrance floor, contains the pharmacy, x-ray department, and laboratory, which are available for the use of the occupants. The six floors above contain doctors' offices. All upper floors were left completely open except for the central vertical cores enclosing the stairs, elevators, and toilets. A suspended ceiling was installed in a 1 x 4-ft module, which allows a flexible arrangement of office partitions. Lighting fixtures and air conditioning diffusers can be quickly interchanged with acoustical ceiling units when required by new partition arrangements.

Air conditioning is ducted from the basement with high velocity mains to each floor and zoned low velocity distribution on each floor. Outlets with flexible connections permit moving outlets to fit partition changes.

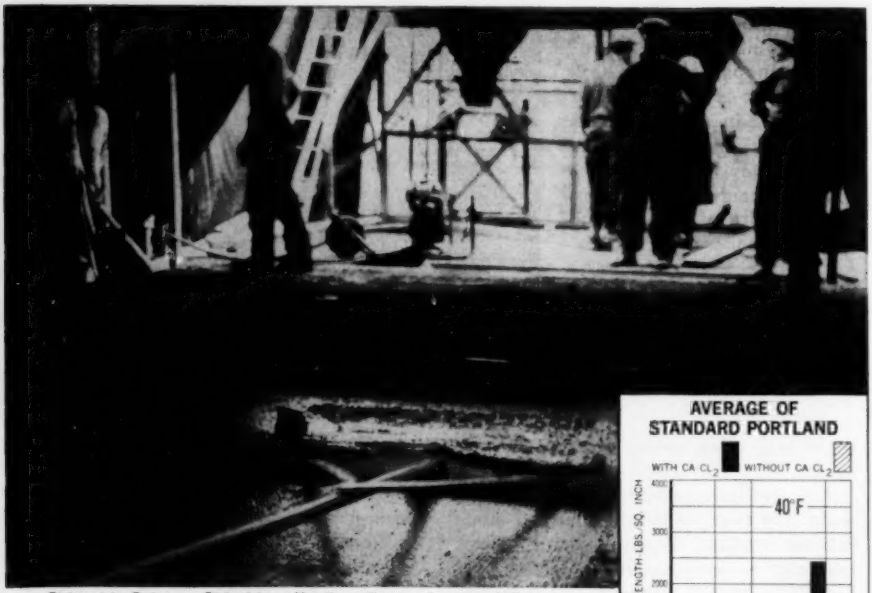
Architect for the building was Mark Hampton, AIA, Tampa. Structural engineers were J. C. Russello and S. L. Barker; mechanical engineer was Charles T. Healy, all of Tampa. General contractor was DeWitt, Furnell and Spicer, Inc., St. Petersburg, Fla. Ready-mixed concrete was furnished by Tampa Sand and Material Co.; expanded shale aggregate was furnished by Georgia Lightweight Aggregate Co., Atlanta. Owner of the building is a corporation formed by a group of Tampa doctors.

Symposium on concrete pavements to be held in Buenos Aires

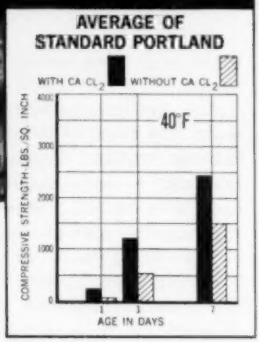
Official and private institutions of Argentina engaged in the highway construction field will sponsor a symposium on concrete pavements in Buenos Aires, Argentina, November 21-26.

While the meeting was planned looking at the national plans for airport and highway development, its international character will permit attendees to become acquainted with the research progress of engineering and technical leaders of every country.

ACI members have been extended an invitation to participate in the symposium. Persons interested in presenting papers or monographs at this meeting are asked to contact the president of the executive planning committee, Juan F. García Balado, Simposio Pavimentos de Hormigon, Calle San Martin 1137, Buenos Aires, Argentina.



Columbia Calcium Chloride, effective all year, is especially beneficial in concrete when temperatures drop below 70°.



The increase in early strength from the use of Calcium Chloride is clearly shown in the above chart.

Chilly fall temperatures holding up your concrete work? COLUMBIA CALCIUM CHLORIDE keeps jobs on schedule

Don't be faced with the problem of job slowdown this fall. When chilly day temperatures drop, keep your concrete jobs moving by adding Columbia Calcium Chloride at the rate of 2% by weight of cement for these time-and-profit saving benefits:

Higher Early Strengths—at 40°F, you get 2-day strength in 1 day, 7-day strength in 3½ days. There's no cold weather lag in strength gain; your job stays on schedule. (See chart above.)

Faster Initial Set—Under normal conditions, Columbia Calcium Chloride cuts initial set time from 3 hours to 1 hour. At lower temperatures, the effects are even more pronounced. You get finishers on and off the job faster, avoid costly overtime.

Early Form Removal—Rapid strength gain permits removal and re-use of forms in about half the normal time.

Are you getting these Columbia Calcium Chloride cold weather benefits on your concrete jobs? If you aren't, why not analyze your batching procedure or contact your ready mix supplier today? Specify Columbia Calcium Chloride.

Let our Columbia Calcium Chloride engineers help you save money on your concrete work this season. For their assistance, just contact our District Sales Office nearest you or write our Pittsburgh Office.

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Economic forecast probes U. S. economy to 1975

IN ALL MEASURES OF OUTPUT, the economy of 1975 will be nearly the equivalent of two American economies of today. The gross national product—the value of all goods and services produced—now totals approximately $480 billion a year. By 1975, the gross national product will reach $925 billion, measured in terms of today's dollar.

This is one of the forecasts made in an economic study "The Years Ahead: 1960 to 1975," recently prepared for the American-Marietta Co. The company serves a wide range of American industry, including the construction field, and is the nation's seventh largest producer of cement.

The report puts the spotlight of economic research upon three salient factors: (1) a continuing and rapid increase in population; (2) a new and greater period of construction that will dwarf the post-war building boom, and (3) an era of unprecedented technological progress.

By 1975, the annual expenditures for new construction, maintenance, and repair will total $137 billion, it is predicted.

Construction for business and industry—In 1960, industry and business, including public utilities, will spend an estimated $36 billion for new plant and equipment. By 1975, this outlay will have increased to $66.7 billion annually.

Roadbuilding — A continuous increase in the number of automobiles will require accelerated highway construction programs. In the last 12 years, the number of automobiles has doubled, but highway miles have been increased only 4 percent since World War II. Fifteen years from now, there will be 111 million motor vehicles in use, compared with 71 million today. In 1975, $11.3 billion will be spent on highway construction.

Need for new homes—More houses must be built in the next 15 years than in the last 30 years. The next 15 years will require the construction of 22 million houses, and suburban home building alone will represent $400 billion in construction.

During the next 15 years, our economic growth will require spending the enormous sum of $1540 billion for construction materials, services, and labor, according to the American-Marietta report. "In itself, this will constitute one of the major forces of growth within the economy."

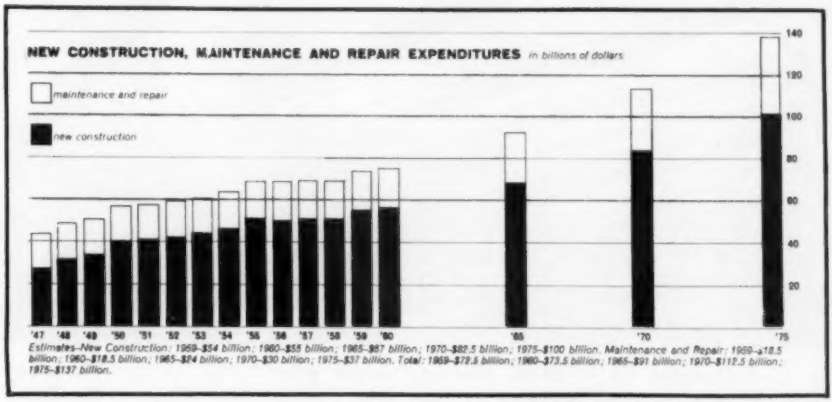

NEW CONSTRUCTION, MAINTENANCE AND REPAIR EXPENDITURES in billions of dollars
maintenance and repair
new construction
140
120
100
80
60
40
20
'47 '48 '49 '50 '51 '52 '53 '54 '55 '56 '57 '58 '59 '60
'65
'70
'75
Estimates–New Construction: 1959–$54 billion; 1960–$55 billion; 1965–$67 billion; 1970–$82.5 billion; 1975–$100 billion. Maintenance and Repair: 1959–$18.5 billion; 1960–$18.5 billion; 1965–$24 billion; 1970–$30 billion; 1975–$37 billion. Total: 1959–$72.5 billion; 1960–$73.5 billion; 1965–$91 billion; 1970–$112.5 billion; 1975–$137 billion.

Chart courtesy American-Marietta Co.

New construction, maintenance, and repair expenditures—1947-1975

Lightweight concrete saucer-like Roof for jet terminal

■ THE SINGLE MOST distinctive feature of the new Pan American World Airways passenger terminal, New York International Airport (Idlewild, Long Island), is its immense saucer-like roof. Over 4 acres in area, and large enough to cover Yankee Stadium, it seems to hover gracefully over the central terminal structure and extends out to protect jet planes and passengers from the weather.

Translating this roof concept into steel and concrete called for bold and unorthodox answers by designers and builders alike. Tippetts-Abbett-McCarthy-Stratton, New York, were engineers and architects for the structure with Ives, Turano and Gardner, New York, as associate architects. Turner Construction Co., New York, was general contractor.

Basically, the elliptical-shaped roof is like a huge wagon wheel with 32 spokes. The hub is fixed by a central anchorage, but the actual support of the spokes is a series of heavy piers midway out to the rim (Fig. 1).

Cantilevering 114 ft beyond these piers, the canopy portion of the roof, which shelters six jet liners at once, is framed into the 32 spokes. These spokes are welded steel girders each prestressed with six heavy cables, which pass over a stanchion mounted on the girder directly over the supporting pier underneath.

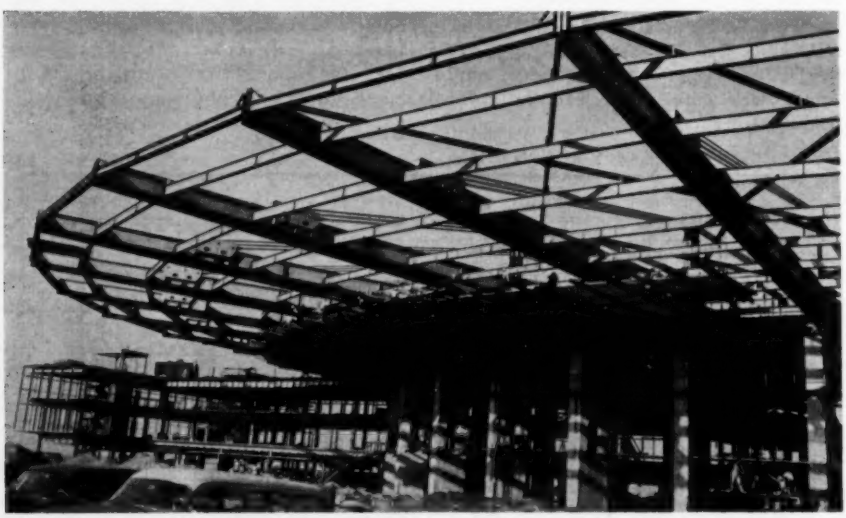


Fig. 1—Framework for elliptical shaped "wagon wheel" roof of Pan American World Airways terminal building at New York International Airport. The falsework to be used to place the roof formwork has been hung from the steel framework. Note the heavy piers supporting the "spokes"

Roof design

Two requirements were paramount in the roof and ceiling construction: holding roof weight to the minimum (with accompanying savings in supporting structure and foundations); and achievement of sound absorption along with decorative potential.

Except for a limited area of the roof slab inside the ring of roof-supporting piers where load requirements made bar reinforcement more practical, wire fabric was specified for the slab, with its attendant reduction in weight of reinforcement. In the design of the concrete canopy, the snow load was assumed to be 40 psf.

The designers specified a lightweight aggregate (Norlite) concrete, weighing 100 lb per cu ft.

In solving the sound absorption, decorative requisite, the designers specified that Pittsburgh Corning acoustical cellular glass insulation in 3-in. thick blocks be installed as the ceiling material. This material weighs 2 lb per sq ft and is attractive when painted.

Roof construction

Converting the plan into actuality was a test of the construction savvy of the Turner organization. With the wagon wheel framework in place, and construction underway on the central building, construction of the roof slab itself and ceiling became the next problem.

Probably basic to the subsequent, smooth progress of the job was the decision to suspend from the roof reusable falsework for all the roof construction and to *hang* all formwork for the concrete roof slabs (Fig. 1). The falsework was made up of sections approximately 10 ft square, usually a dozen or more planks nailed to two heavy timbers. With manual winches at all four corners, the workers then hoisted themselves up on the section to the desired elevation, and tied the section in (Fig. 2).

Fig. 2—Workers hoist themselves up on section of falsework to level of previously placed scaffolding, just under the framework. The suspended falsework simplified erection of the formwork for the lightweight concrete roof. All falsework and formwork was suspended from steel frame

Enough falsework was made up to work on 18 bays of the roof at once, and as work progressed around the elliptical wagon wheel, sections were removed from the far end and hoisted up to join the forward end of the work.

With the falsework in place, the carpenters then placed the formwork for the roof slab from underneath, the entire formwork system being supported by hangers from the radial girders and purlins which brace and stiffen the roof framework. Fifteen sets of forms, enough for nearly half of the 32 wedge-shaped bays in the roof, enabled work "on top" to proceed in sequence without any delay. After concrete in a bay had set and cured, formwork was stripped, and leap-frogged ahead to the end of the job, following up on the newly placed falsework.

Fig. 3—Lathers cut out sheet of welded wire fabric to fit around drain in roof structure. Fabric rests on flange of purlin (see to left of workers) and "drapes" down to rest on 1 in. high chairs on cellular glass block, which forms ceiling of structure. Note lapping of fabric sheets (left) to assure continuity of reinforcing. Pigtailed reinforcing next to radial girder (right) will be tied into fabric sheet. Note hangers around purlins which support falsework below. The concrete hoist can be seen in the center background and a runway for concrete buggies on the right

Photos courtesy Wire Reinforcement Institute

"On-top" operations followed this sequence: First step was to place the concrete encasement of the radial girders (or spokes of the wheel), formed previously from below, and into which had been placed bar reinforcing, pigtailed out for later tieing in to the roof slab.

Then, on the newly placed plywood forms, workers laid 12 x 18-in. blocks of 3 in. thick cellular glass insulation, with the blocks butted tightly against each other and completely covering the area of the roof slab between girders and purlins. Over this, roofers then applied a coat of asphalt mastic waterproofing adhesive.

Next, the lather crew moved onto the job, first placing 1 in. high chairs, and then laying in sheets of 4 x 6-000/2 welded wire fabric (Fig. 3). The sheets were generally 10 ft wide and up to 16 ft long, the distance between purlins in the outer portions of the wedge-shaped bays.

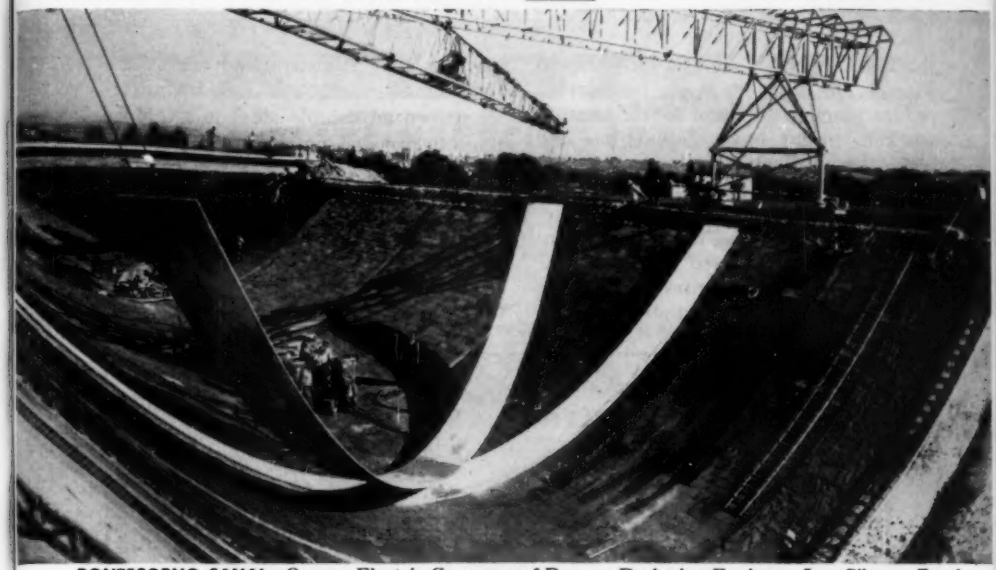
In these outer sections the fabric was placed so that the heavier wire, No. 000 gage, spanned the distance between purlins. For the roof slab in the interior parts of the bays, the fabric was placed so that the heavier wire spanned between radial girders. The sheets of reinforcement were placed so that they rested on the flanges of the purlins, and then draped down to the 1-in. chairs on the plywood forms.

In lapping the sheets, care was taken always to nest the heavy No. 000 gage wires in each other, with the lighter No. 2 gage wires of the sheets on top and bottom of the lap. This assured effective continuity of reinforcing and also reduced the cross-sectional area of the lap to a minimum, and permitted optimum concrete cover.

The sheets of fabric were trimmed on the job to fit the angular sides of the roof slab areas, each one being, in effect, a trapezoidal section out of the wedge-shaped bay. To assure continuity of reinforcing from girder to slab, the bar reinforcing previously pigtailed out of the concrete girder encasement was bent to the fabric sheets and tied.

With the reinforcement in place, workers then inserted holding clips in the cellular glass blocks. Three of these galvanized clips, 3½ in. long, with a 1 in. long angular tab on top, were pushed into each block at an angle, thus furnishing a three-way grip on

POZZOLITH makes good concrete better



PONTECORVO CANAL—Owner: Electric Company of Rome • Designing Engineer: Ing. Silvano Zorzi
Contractor: Farsura, both of Milan, Italy.

Flexible concrete plates drape-shaped to line canal bed

New technique uses prestressed POZZOLITH concrete

The Pontecorvo Canal carries water five miles to this hydroelectric project owned by the Electric Co. of Rome, Italy. Because the canal crosses irregular terrain, running partly in fill and partly in cut, a flexible concrete lining able to adjust itself was specified.

Inch-thick plates of prestressed concrete, four feet wide and up to 98 feet long, were used to line the paraboloid canal bed. A gantry crane gripped both ends of each precast plate, causing it to drape into a catenary curve when lifted. Placement and alignment of successive plates in the prepared mortar bed over the masonry base thus required a minimum of skilled labor. The joints were then sealed with non-shrink EMBECO® mortar.

Concrete for this project had to be durable, impervious and economical. To help assure this performance POZZOLITH was specified, because Ing. Silvano Zorzi's previous experience with it showed that POZZOLITH makes good concrete better. Here, POZZOLITH in the mix reduced the total water content and improved all of concrete's desirable qualities, giving less shrinkage, lower permeability and higher strength.

Your local Master Builders field man will be glad to discuss your concrete requirements with you, and show that POZZOLITH concrete is higher in quality and more economical than plain concrete, or concrete made with any other admixture.

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the block by the concrete when it was placed. The clips could not be inserted before placing the steel fabric because workers might have stepped on and bent them, displaced them, or even torn some of the clips loose.

Final step in the roof slab construction was placing the lightweight concrete. The ready-mixed concrete was hoisted up at the central anchorage of the structure, and distributed by buggies over runways laid across the radial girders, purlins, and temporarily placed adjustable steel joists. More than 4000 cu yd of concrete went into the roof slab.

New York Roofing applied the mastic waterproofing; Hickey Concrete furnished the ready-mixed concrete; Larry Gilmore was job superintendent for Turner Construction Co.

LOOKING AHEAD

Sept. 15-16, 1960 — 8th Annual Engineering Management Conference, Morrison Hotel, Chicago

Sept. 27-30, 1960 — 6th Annual Convention, Prestressed Concrete Institute, Statler-Hilton Hotel, New York, N.Y.

Oct. 10-14, 1960 — Annual Convention, American Society of Civil Engineers, Hotel Statler, Boston, Mass.

Oct. 13-15, 1960—75th Anniversary and National Convention, Tau Beta Pi, Lehigh University, Bethlehem, Pa.

Oct. 31-Nov. 1-2, 1960—13th Regional Meeting, American Concrete Institute, Pioneer Hotel, Tucson, Ariz.

Nov. 3-4, 1960—Annual Meeting, National Slag Association, Hotel Mayflower, Washington, D.C.

Nov. 14-15, 1960—Prestressed Concrete Conference, University of California Extension and the University of California Department of Engineering, Biltmore Hotel, Los Angeles, Calif.

Nov. 14-19, 1960 — Public Works and Municipal Services Congress and Exhibition, Olympia, London, England

Nov. 21-22, 1960—Prestressed Concrete Conference, University Extension and the University of California Department of Engineering, Sheraton-Palace Hotel, San Francisco, Calif.

Nov. 21-26, 1960—Symposium on Concrete Pavements, Buenos Aires, Argentina

Dec. 12-15, 1960—First Industrial Building Exposition and Congress, New York Coliseum, New York

Jan. 5-7, 1961 — FIP-RILEM Symposium on Injection Grout for Prestressed Concrete, Norges Tekniske Hogskole, Trondheim, Norway

Jan. 17-19, 1961—44th Annual Convention, National Crushed Stone Association, Hotel Americana, Bal Harbour, Fla.

Jan. 23-26, 1961—31st Annual Convention, National Ready Mixed Concrete Association, Hotel Americana, Bal Harbour, Fla.

Jan. 30-Feb. 2, 1961—12th Biennial Concrete Industries Exposition, National Concrete Masonry Association, Cobo Hall, Detroit, Mich.

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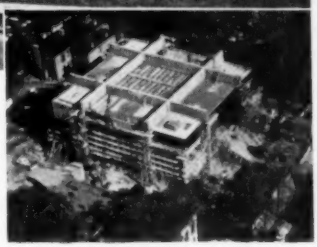
Architect: Philip Johnson Associates, New York City
Structural Engineer: Lev Zetlin, New York City
Contractor: George A. Fuller Co., Boston, Mass.



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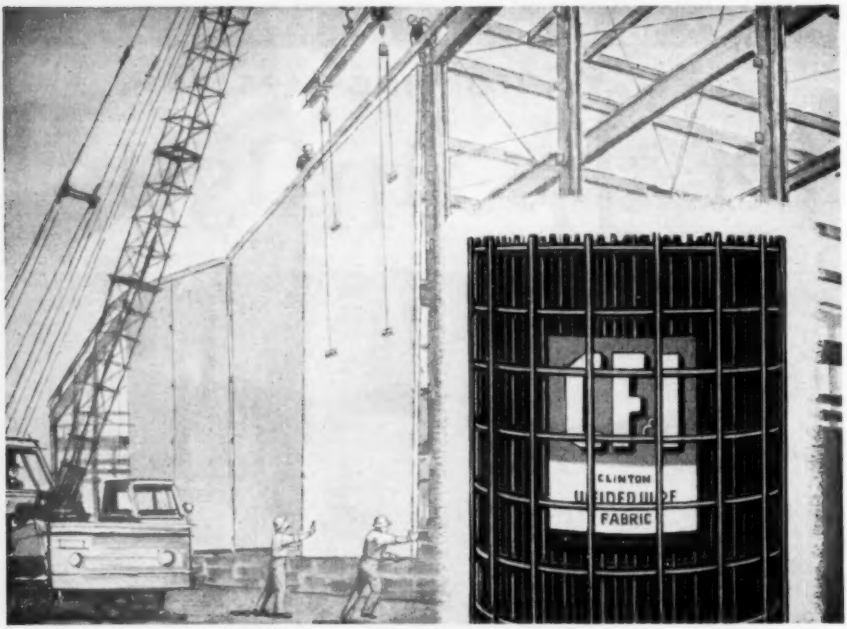

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Positions and Projects

Jelley engaged in Air Force project

Rear Admiral Joseph J. Jelley, USN, Ret., one-time chief of Bureau of Yards and Docks, is serving as Colorado Springs representative of the architect-engineer named to design the Air Force Academy stadium.

Air Force Academy Foundation, Inc., with $1 million of the $3.5 million it needs for the stadium already raised, has engaged Praeger, Kavanagh and Waterbury, New York. Architect Gordon Sweet of Colorado Springs is also associated, and Skidmore, Owings and Merrill, Chicago, will serve as consulting architects.

UA to host conference on data processing

The Numerical Analysis Laboratory at the University of Arizona will act as host to a conference on data processing problems scheduled tentatively for Mar. 16-17, 1961, according to an announcement by A. Wayne Wymore, director of the laboratory.

Interested persons in education, industry, and government are being invited to the meeting which will have the theme: "Discussions of data processing problems in engineering and scientific research." The conference program will include sessions on experiment design, systems design, and on equipment and demonstration.

Eighth Annual Engineering Management Conference

Over 1000 men in the engineering management field are expected to attend the Eighth Annual Engineering Management Conference at the Morrison Hotel in Chicago, September 15-16. The subject of the conference will be "Managing Tomorrow." This conference is sponsored by seven professional engineering societies: American Institute of Electrical Engineers, American Institute of Mining, Metallurgical and Petroleum Engineers, American Society of Civil Engineers, American Society of Mechanical Engineers, American Institute of Industrial Engineers, Institute of Radio Engineers, and American Institute of Chemical Engineers.

Hulsbos Joins Lehigh faculty

C. L. Hulsbos has joined the Lehigh University faculty as research professor of civil engineering and chairman of the Structural Concrete Division of Fritz Engineering Laboratory.

Dr. Hulsbos has been a member of the faculty at Iowa State College for the past 14 years. Prior to joining the staff there, he served with the Iowa State Highway Commission and on the staff of American Bridge Co.

IASS plans two shell structure colloquims

The International Association for Shell Structures, in its aim of furthering the development of shell structures, is arranging two colloquims dealing with this construction technique.

The first assembly, in cooperation with the International Union of Testing and Research Laboratories for Materials and Structures (RILEM), will deal with shell research and is to be held in Delft, The Netherlands, from Aug. 30—Sept. 2, 1961. A. E. Haas, professor, Technische Hogeschool, Delft, is chairman of the organizing subcommittee and A. L. Bouma, professor, Technological University, Delft, is secretary.

The second colloquim will be held in Brussels, Belgium, from Sept. 4-6, 1961. It will deal with simplified calculation methods for shell structures. A. Paduart, professor, Universite Libre de Brussels, Belgium, is chairman of

the organizing subcommittee for the Brussels meeting and Professor Dutron, 127 Av. Adolphe Buyl, Brussels 5, is secretary.

Persons interested in these colloquims may contact the secretaries of the organizing subcommmittees.

Aubrey opens consulting office

W. H. Aubrey, formerly associated with Phillips Structures, Ltd., London, has opened his own consulting office in Coulsdon, Surrey, London.

"Portable" bank vault

Last year Bank of America opened a temporary branch office in Sacramento, Calif., which featured a "portable" vault. The 9½ x 9½ x 12-ft lightweight concrete vault was precast by Ben C. Gerwick Co., San Francisco, at a plant about 80 miles from the erection site. The 12-in. walls were reinforced each way with #4 bars; a 1-in. cover was provided in each face. Concrete was proportioned to provide a strength of 2500 psi at 28 days and a density of 110 lb per cu ft. Forms were plywood to provide an architectural finish.

The vault, which weighed about 30 tons, was transported on a flat bed truck trailer. Installation was accomplished with block and tackle and wood roolers. Handling bolts had to be removable to make the 1-in. clearance through the service entrance of the remodeled quarters. When branch growth warrants, the vault can be moved to a larger building.

McCarthy reappointed to Indiana registration board

James A. McCarthy, professor of civil engineering at the University of Notre Dame, Notre Dame, Ind., was recently reappointed for the third time to the Indiana State Board of Registration for Professional Engineers and Land Surveyors. The appointment was made by Governor Handley of Indiana for a 4-year term.

Professor McCarthy, an ACI member since 1940, is currently a member of the Institute Board of Direction, and a member of the ACI Standards Committee and Committee 115, Research.

Tunisian building tradesmen on tour

Twelve Tunisian building tradesmen recently completed a 46-day, 12-city tour of this country to observe American construction practices.

While in St. Louis, Mo., the group were the guests of Western Waterproofing Co., where they were instructed in various methods of building restoration and weatherproofing.

The purpose of the trip was to study basic residential and commercial building, and the relationships among the municipal codes departments, architects and engineers, contractors, suppliers and building trades.

Mallon promoted to rank of associate professor

Merrimack College, North Andover, Mass., has announced the promotion of Arthur H. Mallon to associate professor in civil engineering.

Professor Mallon joined the Merrimack faculty in 1955 as an assistant professor. Prior to that time he was senior instructor in applied mechanics at Wentworth Institute, Boston, and before that a member of the faculty of the University of Missouri, School of Mines and Metallurgy, Rolla, Mo.

Trade school offers engineering aide course

The T. H. Harris Vocational-Technical School, Opelousas, La., is offering a 2-year course of study for the construction engineering aide at subprofessional level. The program is designed to produce technically trained engineering aides to assume responsibilities where college graduates are not necessarily required.

The course covers essentials for construction engineering such as basic mechanical drawing, vocational and technical mathematics, trigonometry, plane surveying, estimating, and map drafting. Other subjects in the curriculum include a practical course in concrete including classroom and laboratory work. Soils, paving materials, and materials and methods of construction are also covered in detail.

Glenn Madere, school director, states that it has taken several years and considerable assistance from advisory committees, prospective employers, and other interested individuals to get this course started.

Raeder joins faculty at University of Hawaii

Warren Raeder, long-time professor and head of the department of civil engineering, University of Colorado, Boulder, has joined the staff at the University of Hawaii, Department of Engineering, Honolulu, Hawaii.

Sprague heads up concrete research at Lock Joint Pipe

John C. Sprague recently joined the research staff of Lock Joint Pipe Co., Wharton, N.J., as head of its concrete section, after his retirement from the Corps of Engineers, U. S. Army.

Mr. Sprague has more than 30 years of well-rounded experience in the general field of concrete. He is currently a member of ACI Committee 609, Consolidation of Concrete.

The wet-mix REFRACT-ALL GUN cuts costs & waste, speeds up patching & repairing!

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Substructures Rise for Throgs Neck Bridge

■ CONCRETE PLACEMENT in substructures requiring various dimensions to support sweeping approaches to Throgs Neck Bridge, under construction in New York City, was simplified for Merritt-Chapman & Scott Corp. through use of specially designed steel forms. Both the Bronx and Queens approaches including sloping curves—with varying grades and banks.

These variations resulted in a difference in the supporting pier dimensions, and continual adjustment of forms to place concrete. Adjustability of the special forms, designed and fabricated by Blaw-Knox Co., helped reduce concrete forming costs considerably—including a 50 percent reduction in time normally required for finishing operations.

M-C&S built 38 piers under approaches to the $130 million span at a cost of $7 million. The bridge, 92 ft 8 in. wide and with an 1800 ft long center span, when completed early in 1961, will extend 3 miles across Long Island Sound, linking the Connecticut Thruway, Bronx Expressway, and the Belt and Hutchison River Parkways.

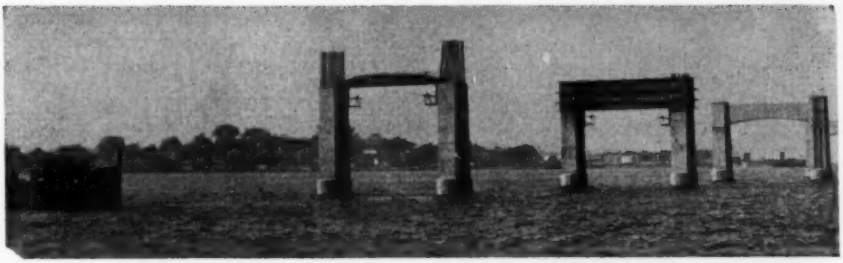
A barge-mounted crane is used to position a cap form. Just behind are a pair of shafts in the concrete curing stage

Although each pier differs in size, all contain an average of 1250 cu yd of reinforced concrete. Construction of each pier included below water level concreting of a tremie, distribution block, and pedestals from which from two to five shafts rise through water level to as high as 107 ft above the water. The shafts are joined by a 112 ft long horizontal concrete cap, which will support the roadway. M-C&S crews adjusted the special forms for concreting the sloping rows of piers, and halved the time, in comparison with wood forms, for form positioning and stripping. Only the "seams" required finishing. This reduced finishing operations.

Substructure pilings were driven 60 to 160 ft. First concrete was placed in tremies and distribution blocks, atop the pilings and below water level. These structures required an average of 400 cu yd of concrete.

Then, the ¼ in. thick heavy forms—reinforced with H-beams, angles, and channels—were bolted together to concrete the pedestals, shafts, and caps. M-C&S operated with four sets of pedestal and cap forms and three sets of shaft forms.

Three to 5 days were required to position forms, place and cure concrete, and strip and reposition forms for placing concrete in another set of pedestals, shafts, and cap. Each pedestal ranges from 12 to 14 ft high and contains an average of 250 cu yd of concrete. Maximum shaft height is 107 ft above the water, and required the maximum lift of 50 ft. The storage pier caps measure 112 ft long, and are 7 ft wide and 12 ft deep. Cap forms were braced between shafts with steel



Steps in construction (left to right) of substructure for Throgs Neck Bridge. First, pilings were used to form a dam. Water was pumped out and forms positioned for concrete placement. Then, two shafts have cured and positioning of cap forms is underway. Next, cap forms are set for concrete placement. Finally, pedestals, shafts, and cap have been concreted and cured

beams prior to concrete placement. These beams remained in position during the curing period.

More than 50,000 cu yd of concrete were used to build the approach substructure, which totals 9000 ft long. Concrete was supplied from a steam-powered floating concrete plant capable of batching 120 cu yd per hr. Concrete for the pedestals and parts of the shafts was transported via buckets that were raised in a tower and then dumped into a hopper-mounted chute to the forms. To move concrete to working level for the upper section of shafts and caps, M-C&S employed a crane-bucket combination.

Although concrete specifications required a strength of 3000 psi after 28 days, tests showed 4500 psi after only 4 or 5 days.

M-C&S also built concrete foundations for the tower pier supporting bridge cables for the center span.

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1239 MILES TO KANSAS CITY
1450 MILES TO NEW ORLEANS
2360 MILES TO MIAMI
937 MILES TO DALLAS
330 MILES TO LOS ANGELES
638 MILES TO SAN DIEGO

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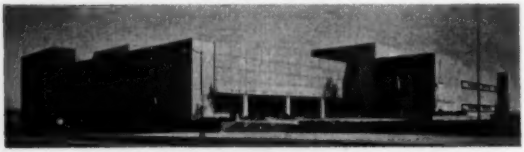
if you're going to—

Tucson, Oct. 31-Nov. 2

13th ACI Regional

Meeting



Cleveland Press Building—Press room occupies lower portion of entire left wing

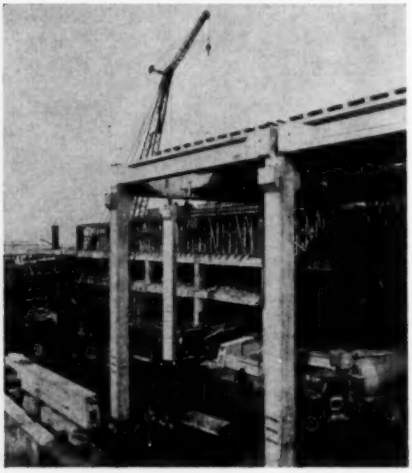
Cleveland Press Building Features Precast Prestressed Members

THE NEW $12,000,000 Cleveland Press Building, Cleveland, has been described as "probably the most efficiently laid out plant ever constructed for a major newspaper in the United States." A massive structure measuring 325 ft x 300 ft, it has 300,000 sq ft of usable area which permits the inclusion of all operating departments of the Press in one integrated building. The building was designed and erected by The Austin Co., Cleveland.

More than 2 years of research and planning went into the design of the building. The latest architectural and construction techniques were used in the reinforced concrete building, which also features generous treatment of limestone, brick, and glass. Precast and prestressed concrete structural members were used for the press room framing in the interest of speed and economy; the balance of the building frame is of reinforced cast-in-place concrete.

The three-story press room is large enough to eventually accommodate 72 high speed presses; 44 presses are now in operation.

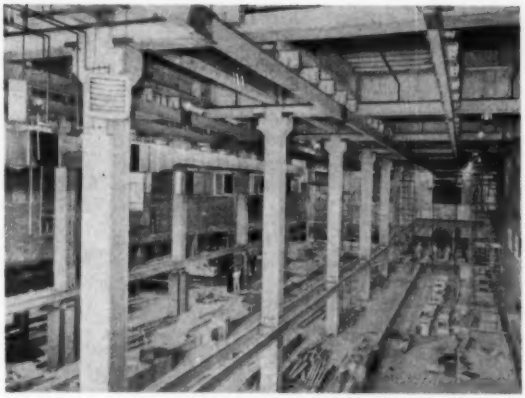
Precast and prestressed concrete members were used in the press room section of the building to minimize the possibility of interference by heavy winter weather. Besides offering important time-saving advantages, these precast members were said to be more economical than cast-in-place concrete.



The crane in operation here is setting one of the 35-ft, 11-ton precast columns which in turn support 30-ft prestressed concrete girders and beams. Haunches in the precast columns support the 12-ton girders. One of these girders can be seen on the ground, just beyond the column on the extreme left, while another, near the center of the building, is alongside several of the shallower 4½-ton beams. Two beams can be seen in position on top of the columns in the immediate foreground, while others directly behind them are supported by the prestressed girders. Some of the precast concrete channel slabs for the mailing room floor are in place on top of the beams

Press room in final stages of construction

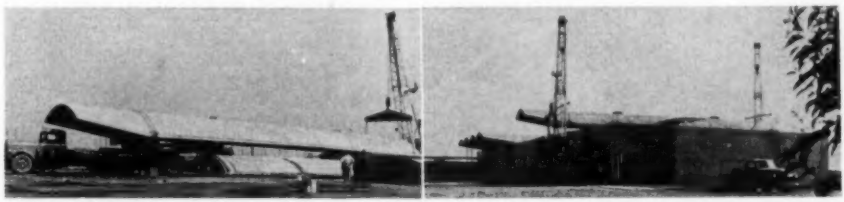
Photos courtesy The Austin Co.



Because of the 35 ft clear height, the formwork and shoring for monolithic concrete, especially columns, would have been unnecessarily expensive.

The 35-ft, three-story, 11-ton precast columns support 30-ft prestressed concrete girders and beams. Haunches in the precast columns support the girders which weigh 12 tons a piece. The shallower beams weigh 4½ tons; 14-ton crane runways are suspended from beams along each aisle. These cranes are used principally to bring in presses or take them out. The crane runways are suspended from the precast beams by specially built brackets that fit around the lower flanges of the precast beams.

The mailing room, which will be located above the press room, will have a floor of precast concrete channel slabs. These 2-ft slabs range in length from 9 to 11 ft and are placed flush with the prestressed girders. They were topped by cast-in-place concrete which tied the various precast structural elements together.



LARGE Precast Barrel Shells . . . are being used for the San Mateo County Commercial and Exhibit Building, in San Mateo County, Calif.

Twenty-two barrel vaults, each 18 ft x 97 ft and weighing 40 tons, will cover the exhibit hall wings of the U-shaped buildings. The barrel vaults are 3-1/2 in. thick except at the arch adjacent to the supports where they vary from 3-1/2 to 10 in. in thickness. The barrel spans 80 ft clear between columns and cantilevers 14 ft 9 in. on one end and 3 ft at the other end. The barrels are cast of lightweight aggregate concrete.

Janssen, Daseking and Keller, Menlo Park, Calif., are the architects for the building. Isadore Thompson, consulting engineer, San Francisco, was the structural engineer for the exhibit hall.

Dundee Cement dedicates $25 million plant

Dedication of the new $25 million Dundee Cement Co. plant at Dundee, Mich., recently, opened one of the world's most modern cement producing facilities, and overflowed the little town of Dundee with a stream of guests from here and abroad.

Guests from the cement industry studied the plant layout and operations, observing Dundee's construction which is reported to have cost slightly less than $5 per bbl (annual capacity) as against the customary $10 to $14 among older facilities. They studied the unique layout, too, to discover how Dundee had made provision in the original design to double initial capacity without enlarging the plant structures.

The plant, built with 60,000 cu yd of concrete, stands on a tract of nearly 1600 acres which are underlain by reserves of limestone and clay—the principal components of cement—that will last more than 100 years. It was this wealth of natural resources that led Holderbank Financiére Glaris, with 26 operating cement companies on four continents, to establish its first venture in the United States.

Only gypsum, which forms less than 5 percent of the finished product, and coal for the two kilns, which are said to be the largest in the Western Hemisphere, need to be transported to the plant.

Construction was started in June, 1958, and it took about 18 months to complete the project. All concrete silos and even the secondary crusher building were built by the slip-form method. The 100-ton mills and kiln sections were placed with two, 100-ton cranes. Erection of the mills was scheduled

Dundee Cement Co.'s $25 million plant. Indicated above are: **(1)** quarry containing clay and limestone deposits; **(2)** primary crusher which reduces limestone to size suitable for conveyor; **(3)** cement storage silos 150 ft high with rail and truck loading stations; **(4)** conveyors and pipelines hauling raw materials and coal; **(5)** crushed limestone and coal storage area; **(6)** storage area for 200,000 tons of cement clinker; **(7)** main plant, housing the five largest grinding mills in the world;**(8)** two kilns 460 ft long—largest in the Western Hemisphere; **(9)** electrostatic precipitator area, where more than 99 percent of all impurities are removed from exhaust before discharge through stack 350 ft high; **(10)** main office building

together with the structural steel, as the roof could only be placed after the mills were installed. By the end of September, 1959, all buildings were enclosed and cement production was under way by the end of the year.

As the plant went up, the excavating machinery on the adjacent quarry area dug into the earth to strip away the 20-ft layer of overburden covering the rich clay and limestone deposits. By the time the plant was ready for operation, the quarry, including a primary crushing mill built below the surface of the earth, was ready to produce up to 5000 tons of limestone and 2500 tons of clay each day.

The cement storage silos are arranged so that railroad cars can be serviced at one loading platform and trucks at another. A few miles away, in Monroe, Mich., a shipping center has been constructed alongside the Raisin River, where marine shipments can originate for transportation over the Great Lakes. Distribution terminals have been built or are being constructed by the company at Chicago, Cleveland, and Cincinnati.

The plant was designed and erected under the direction of the engineering department of St. Lawrence Cement Co., also a subsidiary of Holderbank, in Clarkson, Ont., Canada, John Hiltemann, chief engineer.

Darin and Armstrong, Inc., Detroit, installed the $10 million worth of machinery in the plant, and Harlan Electric Co., Detroit, installed the electrical components. The Nicholson Co., New York, constructed the storage silos, the cooler pit, and much of the building proper. Numerous other contractors were involved in the huge undertaking.

Watson joins Prescon Caribbean division

J. Raymond Watson, formerly instructor of civil engineering, University of Puerto Rico, has been appointed chief engineer of Prescon Caribe, Inc., San Juan, Puerto Rico.

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Competition is keen in construction

"The 1960 economic rebound lacks the bounce predicted for it by New Year's Day prophets," remarks GeoSurveys in its periodic review of materials for construction.

"What is needed more than easier money is protection from the fly-by-nights who are doing the country's largest industry no good," said the review. Reliable, old-line contractors are plagued by inexperienced newcomers who underbid them, get the business, and go broke. Last spring the Department of Defense was left with a dozen half-completed construction projects on its hands when an overextended contractor went bankrupt.

For a variety of reasons, construction in its broadest connotation is off, compared with 1959, when $54,258 million was spent for new building. In the first quarter of this year, total construction outlays were down 2 percent from the 1959 level. Public building was off 14 percent and new housing starts dropped 17 percent, to a seasonally adjusted rate of 1.15 million units. Since March the situation has improved.

Suppliers of construction materials prospered in 1959, and seem to be doing well thus far in 1960, noted GeoSurveys. Last year seasoned contractors anticipated the steel strike and stockpiled their steel requirements. Facts and figures are not available, but the expectation of delays in deliveries of structural steel prompted many a builder to turn to reinforced concrete. The rise in imports of reinforcing steel during 1959 was noteworthy.

Cement production, which in 1958 rose 5 percent over the preceding year, jumped another 9 percent in 1959. Output was in excess of 330,000,000 bbl. Riding in on the crest of the wave, the Swiss firm of Holderbank Financiére Claris, S.A., opened a new $25 million plant at Dundee, Mich. Quarrying operations started last November, and by January two giant kilns were in full operation.

Demand for cement has eased, partly because of the lag in road building. Producers figure sales will drop 3 to 4 percent in 1960, and keep competition is anticipated. Nonetheless, Lone Star Cement Corp. is adding a 1,000,000-bbl kiln to its Norfolk plant, and Medusa Portland Cement has started a $70 million, 10-year expansion program that will add 2,000,000 bbl to its productive capacity.

One trend became pronounced in the building trades in the course of 1959, said the review. Moves toward integration were evident in the purchase of American Rock Wool Corp. by U. S. Gypsum; of Calaveras Cement Co. of San Francisco by Flintkote Co., manufacturer of roofing and siding materials. The Warner Co. has diversified its business in aggregates by acquiring Atlantic Prestressed Concrete.

"Percentagewise," noted GeoSurveys, "the volume of construction fluctuates within moderate limits. Barring a major economic setback, its future is predictable. Its relative stability, however, has drawn many firms, large and small, into one phase or another of the business. The result is a highly competitive situation that is good for the builder but bodes ill for the marginal producer."

University of California to hold prestressed conferences

University of California Extension and the Department of Engineering will present two conferences on prestressed concrete in November in Los Angeles and San Francisco. The 2-day meetings will be held November 14-15 at the Biltmore Hotel in Los Angeles, and November 21-22 at the Sheraton-Palace Hotel in San Francisco.

The program will include such topics as architectural design using prestressed concrete members; coordination of prestressed concrete in total design; project examples; connections; types and use of precast elements; code requirements; inspection; specifications, and fire resistance. Prom-

inent engineers and architects will lecture and take part in panel discussions.

Further information about the meetings may be obtained from the Department of Conferences, University Extension, University of California, Berkeley, Calif.

Steinman awarded 26th honorary award

David B. Steinman, internationally known bridge engineer, was recently awarded his 26th honorary degree.

A long-standing tradition was broken at Albion College, Albion, Mich., when Dr. Steinman was awarded the honorary degree of Doctor of Laws *in absentia*. His paper "The Faith of a Bridge-Builder" was read by W. W. Whitehouse, president of the college. Nine of Dr. Steinman's bridges have been honored in the annual awards for the most beautiful bridges in America.

12th biennial concrete industries exposition

The National Concrete Masonry Association will sponsor the 12th biennial concrete industries exposition Jan. 30-Feb. 2, 1961, at Cobo Hall, Detroit.

Russell E. Eichelberger, general manager, Reimers-Kaufman Concrete Products Co., Lincoln, Neb., has been appointed chairman of the exposition and also chairman of the Concrete Industries Exposition Management Committee.

Meyer promoted by Universal Atlas Cement

E. Bernard Meyer, formerly project development engineer, has been appointed assistant chief engineer for United States Steel's Universal Atlas Cement Division, New York. Mr. Meyer joined Universal Atlas in 1939.

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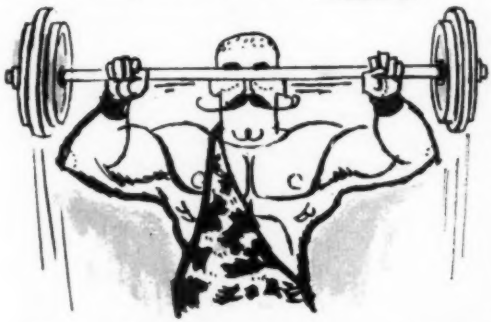

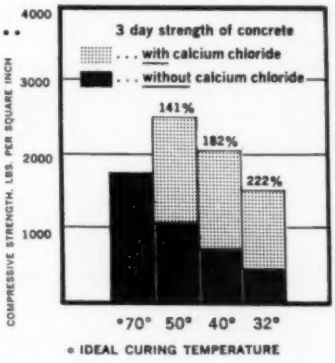

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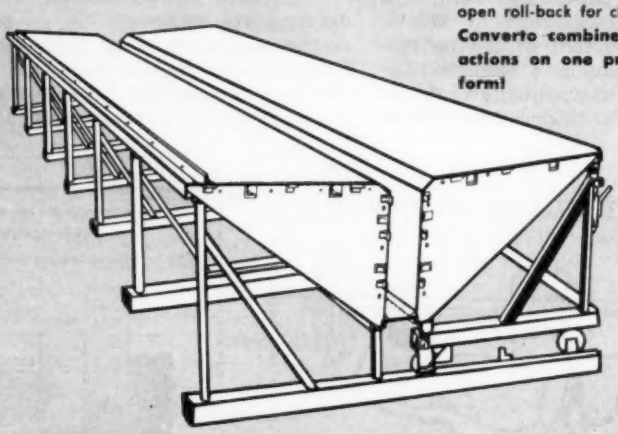

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Sealing Giant Niagara Conduit Joints

SEALING THE DOME joints of the Niagara Power Project's twin conduits with polysulfide-based compound was done this past winter with specially equipped rigs capable of applying the compound at 90 F in all kinds of weather. With orders to continue construction through the winter of the 4 mile long conduits, most of the contractors had to use a good many tricks of the trade to overcome cold weather obstacles.

The New York State Power Authority is building the $720,000,000 Niagara Power Project. Two huge conduits, 66 ft high and 46 ft wide, will carry water from the river above the famous falls to two generating plants 4 miles downstream.

The reinforced concrete conduits are being sealed with polysulfide-based sealant along three longitudinal and regular transverse joints of the arched dome cover. Laid in trenches as deep as 150 ft, the conduits will be covered with fill so that the surface can be utilized for various activities.

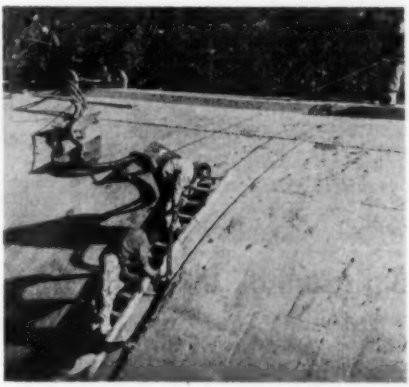
Abutment hinge joints where the arch meets the vertical walls, construction joints at the top of the arch, and joints between each form of the arch all receive heavy beads of the synthetic rubber waterproofing compound. Formulated by Allied Materials Corp. the compound meets special performance specifications written by the Power Authority.

To apply the two-package, chemically cured compound throughout the winter, Railroad Waterproofing Corp., the contractor, had to develop a special rig with heating elements under the reservoir tanks and around the application hose since the compound requires temperatures in excess of 50 F to cure. Using a basic applicating rig, special gas heaters were added at the base of the two component storage tanks. Then electrical heating elements were wrapped around the hose that feeds the applicating nozzle.

By applying the polysulfide sealant at 90 F, there is enough heat in the compound to accomplish curing in a few hours even when temperatures are below freezing. The curing action is exothermic—that is, it produces heat—so that once curing has started at a good rate it will continue even though surrounding air and concrete are absorbing some of the heat.

The polysulfide sealant was applied after sandblasting of concrete joints. The domes for both conduits will require about 1,000,000 lb of polysulfide-based sealant. According to Thiokol Chemical Corp., producer of the polysulfide liquid polymer from which the sealant is made, this is the largest single application of polysulfide sealant ever made.

Railroad Waterproofing is performing this and other waterproofing work

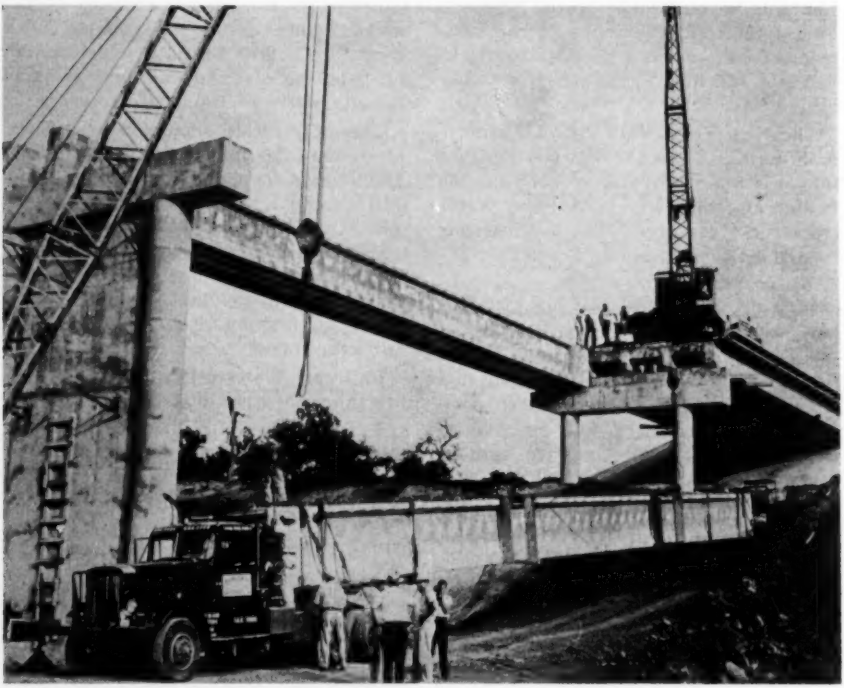


Polysulfide sealant is applied to transverse construction joint on conduit being built for Niagara Power Project. Special rig is seen in left background. Hose is supported on dollies to prevent damage to heating elements that keep the compound hot during winter application

for the entire length of the twin conduits under three prime contractors: Merritt-Chapman & Scott Corp.; Gull Contracting Co., Inc. & L. G. DeFelice & Son, Inc.; Edward Balf Co., Savin Bros. Inc. & D. W. Winkelman Co., Inc. Uhl, Hall & Rich are general engineering consultants for the entire project.

Cottingham and Hale open consulting office

Worth Cottingham, formerly of Wilson and Cottingham, and Isom H. Hale, consulting engineer, have formed a partnership for the general practice of civil and structural engineering under the firm name of Cottingham and Hale, Austin, Tex.



Big beam....big haul

Everything is big in Texas, including its problems . . . when Span Inc., Dallas, produced fifteen 94-ft prestressed concrete bridge beams for a crossing over the Trinity River there was little difficulty in the manufacturing process. However, hauling the beams over 30 miles of twisting side roads to the construction site turned out to be a challenging problem for Dallas trucking contractor R. B. Hunsaker.

After brief tests on handling the huge prestressed members, it was decided to rig two Autocar tractors with rear-steering carriages. The prestressed beams were lifted from the casting forms by two cranes. One end of the beam was placed on the fifth wheel of the Autocar and the other on the rear-steering carriage 94 ft away. The dangerous possibility of side sway when the roadway sloped and twisted was reduced by the installation of sway bracing inside the third-points of the beam.

Delivery was accomplished without incident and the 70,000-lb girders were lifted into position by 40- and 35-ton cranes on each end. Even the Trinity River, which each spring becomes a roaring torrent of water, cooperated during girder installation by confining itself to a comparative trickle.

Photo courtesy Autocar Division, White Motor Co.

Errata

The following correction should be made in "Design of Prestressed Lift Slabs for Deflection Control," by Edward K. Rice and Felix Kulka, which appeared in the February, 1960, ACI JOURNAL.

p. 689—The last line on the page which reads ". . . and live load is $0.30 f_c' \ldots$" should be ". . . and live load is $0.03 f_c' \ldots$"

* * * *

The following corrections should be made in "Distribution of Torsion and Bending Moments in Connected Beams and Slabs," by M. A. Gouda, which was published in February, 1960, ACI JOURNAL.

p. 764—change Eq. (11) to the following corrected form

$$\therefore \mu = \frac{k\,t\,l}{b^2}\left(\frac{t}{S}\right)^{1/2}$$

p. 764—the line immediately under Eq. (11) which reads ". . . where K is determined . . ." should be ". . . where k is determined . . ."

p. 765—in the second equation from the top of the page, change the numerator to read

$$U = 0.08\,\frac{k\,l}{\sqrt{S}}$$

p. 767—in the third equation from the top of the page, change it to read

$$\therefore m = A \cosh cx + B \sinh cx$$

Also on p. 767—change Eq. (14) to read

$$T = \frac{g\,S^2}{12\ c\ \cosh\dfrac{c\,l}{2}} = \text{Sinh } cx$$

* * * *

The following correction should be made in "Method of Assessing Probable Fire Endurance of Load-Bearing Columns," by J. H. Clarke, which appeared in the June, 1960, ACI JOURNAL.

p. 1233—the fourth line from the bottom which reads ". . . 1 1/2 in. of neat gypsum plaster . . ." should be ". . . ½ in. of neat gypsum plaster. . ."

* * * *

The following corrections should be made in "High-Strength Deformed Steel Bars for Concrete Reinforcement," by Sidney A. Guralnick, which appears in this issue, September, 1960, ACI JOURNAL.

p. 248—the column heading "Deformations at load approaching failure load," at the right side of Table 2, should cover only the last three columns at the right side of the table, and the column listed as "Type Failure" should be a separate entry and should not be part of the above heading.

p. 254—in Eq. (3), change that part of the denominator from Q to Q_s and in the line immediately under this change that portion from ". . . . and Q_s refer to . . . . " to read ". . . . and Q_s refer to . . . ."

p. 256—the line immediately above Eq. (4), which reads ". . . . measured stress versus . . . " should read ". . . . measured stirrup stress versus . . . .".

p. 257—For clarification of Fig. 4, the ordinate axis, f_v, is the "measured stirrup stress."

p. 261—change the first two lines immediately under Table 4 to read—"The ratios of actual ultimate moment, M_u, to calculated moment, M_f, given in Table 4 indicate . . . . ."

p. 267—change the last line at the bottom of the page which reads ". . . . II D—1 M." to read ". . . . III D-1M."

p. 269—change the seventh line from the bottom of the page (excluding footnotes) to read ". . . . at failure, V_u, to . . . ."

p. 272—change the fourth line from the top of the page which reads ". . . . value of $\bar{M}/M_f$ is . . . ." to read ". . . . value of M_u/M_f is . . . ."—and in the first and fourth lines from the bottom of the page, change the V_u in both cases to read V_u.

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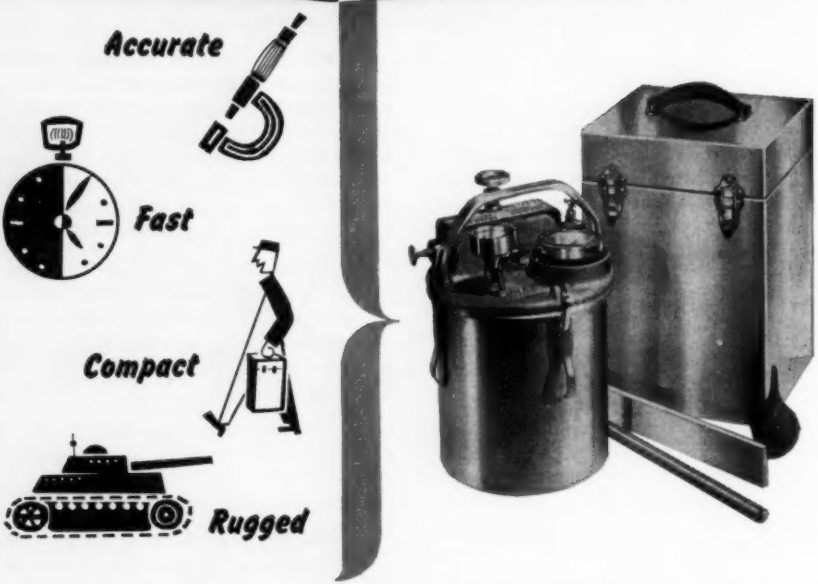

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p. 273—in the second line change "V_u" to "$\overline{V}_u$".

p. 279—in the sixth line from the bottom change "with ⅛-in." to "within ⅛-in."

Hatch elected president of Hercules Cement

Glenn M. Hatch has been elected president of Hercules Cement Co., a division of American Cement Corp., Philadelphia. Mr. Hatch, who has been active in the cement industry for the past 32 years, was formerly vice-president for sales of Hercules.

Wilson joins NSGA-NRMCA staff

The National Sand and Gravel Association and the National Ready Mixed Concrete Association announces the appointment of John Richard Wilson as associate engineer on their joint engineering staff. Mr. Wilson recently received the degree of Master of Science in Civil Engineering from the University of Maryland under the sponsorship of the Stephen Stepanian Fellowship of the National Ready Mixed Concrete Association.

PCA announces new appointments

The Portland Cement Association, Chicago, has announced a number of changes in field personnel.

Robert H. Lochow has been named district engineer for New Jersey with headquarters at Trenton. Charles H. Knight, Jr., succeeds Mr. Lochow as district engineer for the states of Washington, Oregon, and northern Idaho, with offices in Seattle.

Homer A. Humphrey has been appointed district engineer of PCA's Southern New England District, with headquarters in Boston. George K. McCord has been named Wisconsin district engineer with headquarters in Milwaukee.

Improving prospects for building industry

The low point in housing activity has probably been passed, says *The Value Line Investment Survey*. The rate of new dwelling unit starts is expected to turn upward in the second half and climb higher in 1961.

In the public field, road building, postponed by severe weather and an inadequate flow of federal funds, is likely to accelerate over the balance of the year. The rate of commercial and industrial building is expected to continue to show further increases. The Value Line Survey looks for full year new construction volume to come close to the 1959 total.

Total construction outlays for the first 4 months of 1960 were 3 percent less than the volume of the like period in 1959. Highway volume was down 16 percent in the first 4 months.

The drop in over-all construction activity would have been even greater in the first third of the year had there not been marked increases in private industrial (up 35 percent) and commercial (up 13 percent) building and a smaller gain in public utility construction (up 2 percent). Spending on schools (public and private) totalled $0.98 billion, off slightly from $1.04 billion in the first 4 months of last year.

The promise of the decade ahead

New construction activity regularly accounted for about 11 percent of the gross national product in the decade of the fifties. There is every reason to expect this relationship to hold, says Value Line Survey. At least 10 million additional dwelling units will be needed just to house the expected additions to the U.S. population during the next 10 years.

Total new construction expenditures in the $625 billion gross national product economy hypothesized for 1963-65, are projected to $68 billion per year, of which private residential building

will probably account for about $26 billion.

Cement industry prospect

Severe storms across the nation during the first part of 1960 slowed up construction activity, and as a result, nation-wide cement shipments dropped 14 percent below the similar 1959 quarter. Exceptions were New York, Florida, and Arizona where special projects (e.g., dam construction) outweighed the adverse effects of the weather. However, full-year shipments will probably match the record 1959 level.

Another current industry dilemma centers around overcapacity. For the past 3 years, production averaged only 79 percent of capacity versus 92 percent for the preceding 3 years. It is simply a case of capacity expanding at a faster pace than demand. Nor, says Value Line Survey, will this situation be altered within the next year or two. Cement shipments this year will probably closely approximate 1959's 335,000,000 bbl in spite of the poor start. But in the meantime, capacity will grow from about 414,000,000 bbl to about 430-435,000,000 bbl.

Cement is the one material used in almost every type of construction. Demand is certain to increase substantially in the future, as it has in the past, as our population not only expands, but grows more mobile. In addition, the industry is constantly bringing out new forms and new uses for concrete (e.g., prestressed concrete, shell roofs, precast concrete, lightweight concrete). Looking ahead to 1963-65, Value Line Survey foresees consumption of cement expanding at a faster pace than capacity.

Kuklich joins Perlite Institute staff

R. Joseph Kuklich has been appointed director of promotion and public relations for the Perlite Institute, New York.

Walter L. Huber

Long-time ACI member Walter L. Huber, consulting engineer of San Francisco, died recently at the age of 77.

A noted structural and hydraulic engineer, Mr. Huber had maintained his original consulting office without interruption since 1913.

The San Francisco earthquake of 1906, following close on Mr. Huber's graduation from the University of California, afforded him many assignments in the reconstruction of the city. That kindled his interest in earthquake-resistant construction and later resulted in his becoming one of the nation's most eminent authorities in that field.

Mr. Huber's practice covered structural and hydraulic problems. The structural designs for many of San Francisco's office buildings were his work, as was the Union Square underground garage. He was a consultant on California's Central Valley Project for irrigation districts and on Corps of Engineers' flood-control structures in the state.

Mr. Huber was also instrumental in initiating and drafting the California State Dam Control Act.

He had been an ACI member since 1929 and was also active in the American Society of Civil Engineers, serving as president in 1953.

ACPA opens two regional offices

Howard F. Peckworth, managing director of the American Concrete Pipe Association, Chicago, announces the opening of regional offices in California and Georgia.

Jack C. Williams, formerly assistant to the managing director in the association headquarter's office in Chicago, heads up the western region office in Palo Alto, Calif.

Mell Aycock is in charge of the southeastern region office in Atlanta,

Ga. Prior to coming with the association, Mr. Aycock was associated with Augusta Concrete Products Co. of Augusta.

Laclede reinforcing bars carry rolled-in markings

High-strength multirib reinforcing bars manufactured to satisfy the ultimate strength design methods approved by the ACI Building Code, and carrying rolled-in markings for immediate identification, are now being produced by Laclede Steel Co., St. Louis. The new Laclede bars conform to two new standards adopted by the American Society for Testing Materials: A-432, for 60,000 psi minimum yield point steel; and A-431, for 75,000 psi minimum yield point.

The bars are identified for grade by one and two longitudinal ribs, respectively, rolled into the bars at time of manufacture. Laclede bars in other grades also carry rolled-in identification markings. The identification system also includes a number designating the bar size, and the letter "L" for company of origin. Immediate identificationof bar size and strength are easily checked whether the bars are in at the jobsite, stock lengths, or sheared-to-length sizes.

Laclede is the first steel producer to offer comprehensive identification markings in the new reinforcing bars.

Lee elevated to full professor

Seng-Lip Lee, formerly associate professor of civil engineering, Technical Institute, Northwestern University, Evanston, Ill., has been elevated to the rank of professor.

Dr. Lee received successive degrees in civil engineering from Mapua Institute of Technology, Philippines; University of Michigan, Ann Arbor; and University of California, Berkeley. He has specialized in structures and has had 16 papers published in technical journals.

HRB committee on composite pavement design organized

Nineteen of the country's leading pavement designers attended an organizational meeting of the committee on Composite Pavement Design of the Highway Research Board, National Research Council, Washington, D.C., held July 14-15 at Ohio State University, Columbus.

Appointed by the Highway Research Board last March, the committee is set up to encourage research in structural design of composite pavements, which are composed of both bituminous and portland cement materials, and are

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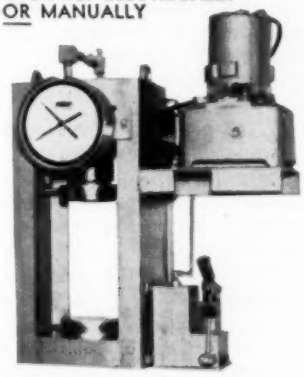

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a layered system of rigid cement and flexible asphalt elements. The committee will collect and disseminate information and assist in the development of theoretical approaches to design procedures.

Robert F. Baker, director of civil engineering research at Ohio State's Engineering Experiment Station, is chairman of the committee.

Dickinson elected president of Calcium Chloride Institute

William E. Dickinson, executive vice-president and chief engineer, has been elected president of the Calcium Chloride Institute, Washington, D.C.

Mr. Dickinson replaces George H. Kimber, who has been president since 1946. Mr. Kimber will remain with the organization in the capacity of chairman of the board.

New appointments by Master Builders Co.

William M. O'Donnell has been appointed export sales assistant to the vice-president for marketing and will head the newly organized export marketing department of The Master Builders Co., Cleveland. In his new post, Mr. O'Donnell will be responsible for coordinating Master Builders sales and services to U. S. architects, engineers, and contractors involved in foreign construction. He joined the organization in 1953.

David M. Burke has been named sales representative for the company's Pittsburgh branch office. Prior to joining Master Builders in 1959, Mr. Burke served on the technical staff of Westinghouse Corp.

Shaw named head of Turner Chicago office

Walter B. Shaw, vice-president, Turner Construction Co., New York, is now in charge of the Turner Chicago office, replacing Clinton N. Hernandez, who has retired.

Honor Roll

Jan. 1-July 31, 1960

Today ACI membership has reached an all-time high. Thousands of persons working in the concrete industry throughout the world need ACI services and ACI needs the best of their thinking. Tell your colleagues about ACI activities! Add your name to the Honor Roll next month.

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Samuel Hobbs	32
Robert P. Witt	17
Walter H. Price	16
Harry Ellsberg	15

Point System

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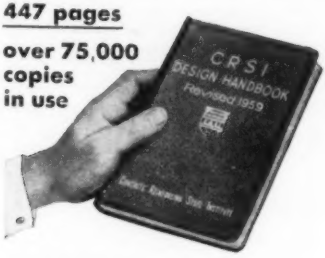

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New Members

The Board of Direction approved applications in the following categories: 57 Individual, 3 Corporation, 18 Junior, and 11 Student, making a total of 89 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,290.

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ANG, CHARLES, Philadelphia, Pa. (Struct. Engr.)
APICELLI, ALFRED R., New Castle, Pa. (Tech. Ser. Engr.)
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BRASELTON, GUY, Corpus Christi, Texas (Pres., Gen. Mgr., Prescon Corp.)
BROCK, MAX, JR., Lawton, Okla. (Mgr., Besco Concrete)
CAMPUZANO, FELIPE PEZO, Guayaquil, Ecuador (Supt., Industrias de Concreto Roca C. A.)
CANNISTRA, STEPHEN A., Alexandria, Va. (Supv. Struct. Engr., Fed. Aviation Agency)
CASAS O., DANIEL, Popayan, Colombia (C. E., Facultad de Ingenieria)
CHAPMAN, ROBERT FULLER, St. Ives, NSW, Australia (Struct. Engr., Cement & Concrete Assn. of Australia)
COVARRUBIAS, JAVIER, Mexico, D. F., Mexico (Mng. Dir., Prescon de Mexico, S. A. de C. V.)
CURLEE, S. H., Clayton, Mo. (Struct. Engr.)
ELSTAD, E. G., New York, N. Y. (Cons. Engr., Severud-Elstad-Krueger-Assocs.)
FJELLVAER, WILLY, Oslo, Norway (Lab. Supv.)
FLETCHER, JOE M., Baton Rouge, La. (Chf. Chem., Ideal Cement Co.)

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. All who have an interest in concrete are eligible for membership. The grades of membership are described below.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

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ACI editors want to know your interests—they welcome suggestions as to what you'd like to see discussed in the ACI Journal and of the possible sources. Please attach a separate note or letter.

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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Expansion bolt

Cram-it wire and rod hanger type expansion bolts, based on the Wej-it reverse cone principle, are now available in a variety of sizes and in all ferrous and nonferrous metals.

For installation, drill hole in the concrete or other nonfrangible material and by hand insert a Cram-it bolt to the desired depth. The unit slides in easily yet will never pull out according to the manufacturer.—Wej-it Expansion Bolt Co., Inc., Kingston, N.Y.

Material-level indicators

Two material level indicators, one designed for hazardous dust locations and one for general purpose use, are being offered by Fuller Co.

The models are identical in operation and dimensions. They contain interchangeable parts and wiring, except that Model SG-4 contains a gasket between the cover and the body, while Model SG-4X has metal to metal surfaces and a paddle shaft bushing machined to Underwriters' Laboratories specifications.

These indicators may be used to indicate the level of pulverized, fine, crushed, or granular materials (maximum 1 in.) in bins and silos. They are used also to control such equipment as feeders, valves, elevators, and conveyors. They exercise this control by starting or stopping the motor driving the equipment when a predetermined material level is reached.—Fuller Co., Catasauqua, Pa.

Patching and resurfacing compound

RIW Epo-Tox is an adhesive, epoxy-type floor patching and resurfacing compound.

The compound consists basically of a blend of epoxy resins, selected mineral aggregate, a catalyst, and color. It can be applied in any required thickness (or thinness) without sacrificing impact and abrasion resistance according to developer. It is quick setting, permitting floors to be returned to normal use after over night curing.

The material is said to be resistant to most industrial acids, alkalies, and solvents, and produces a nonskid granular floor. Slight variations in formulation and in application techniques can be made to achieve a smooth, nonsparking surface.

Available in gray, red, or green, the compound may be applied indoors or out, by trowel or other conventional working tool.—Lucerna Co., Inc., 17 East 45th St., New York 17, N.Y.

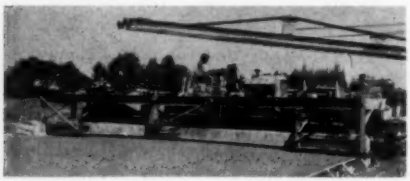
Rebar tension testing machine

Forney's Model TM-100 tension machine is designed especially for pulling reinforcing bars from #2 to #11.

Two ranges are available as standard equipment: zero to 30,000 lb and zero to 250,000 lb. Two sets of gripper blocks handle all sizes. The manufacturer advises that the machine can be arranged to test stranded cable in tension and 6 x 12-in. cylinders in compression, thus making it a multipurpose apparatus.—Forney's Inc., Tester Division, Box 310, New Castle, Pa.

Power driven screed attachment

A specially designed, power-driven screed attachment, mounted at the rear of a concrete spreader, eliminated Harrison Construction Co.'s need for one finisher on many paving projects. The new attachment was also said to eliminate tears or separations in the finish. The screed attachment was built onto the spreader by Blaw-Knox Company.



One of the jobs the 5M screed attachment was used on last summer was a 4-mile, four-lane section of Interstate Route 1 in Ohio. Operating at 24-ft width, two spreaders placed 93,752 sq yd of concrete—supplied from three pavers. Two pavers and a spreader were used to place the first 6 in. of concrete. Reinforcing rods were positioned, and the third paver supplied the top 4 in. of concrete. The second spreader and screed attachment followed the third paver.

Specifications called for 2½-in. crown, varying no more than ⅛ in. over a 10 ft square area. The adjustment was made through a single lever, at one end of the screed, which made corrections through a series of cams. Small crowning variations were instantly and accurately made, allowing proper feeding of material to the front screed of the following finishing machine.

Utilizing an automatic transverse spreading blade to evenly distribute concrete, the paving unit and its special attachment operated on 10 in. high steel road forms.

To mount the screed on the spreader, Blaw-Knox provided a rugged supporting frame. This is a welded double channel beam to support the driving and lift mechanism, and designed to extend behind spreader end trucks. This design also positions rollers for maximum push action on the screed. The screed attachment, controlled by the spreader operator, is powered by the spreader's 6-cylinder gasoline engine—rated at 58 hp at 1620 rpm.—Blaw-Knox Co., 300 Sixth Axe., Pittsburgh, Pa.

Portable concrete tester

A portable concrete tester has been developed to perform compressive strength tests on 6-in. diameter concrete cylinders or 6-in. concrete cubes.

The CT-710-B tester is approximately 38 in. high, 21 in. wide, and 20½ in. deep. It weighs 480 lb and has a load capacity of 125 ton.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

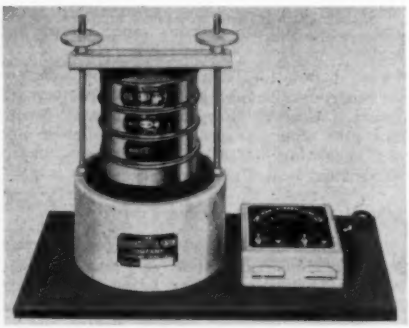
Portable vibrating screen

An 8-in. circular screening unit, designed for research laboratory, production-control labs, and pilot-plant use, weighs only about half as much as similar units.

The screening unit features a vertical-horizontal pulsating action provided by two independently driven rotating shafts operating at adjustable speed differentials. Each motor is rheostat controlled so the pulsating action can be readily adjusted for different materials.

The unit is suited for screening granular, crystalline, powdery, or other dry materials, even through 200, 325, and 400 mesh. It can also be used for screening slurries and fluids.

There is said to be no vibration in the base, so the unit can be set on any table or bench.



Motor operates on DC or 25-, 50-, or 60-cycle AC current. The unit is also available in 12- and 18-in. sizes.—The C. O. Bartlett and Snow Co., 6200 Harvard Ave., Cleveland 5, Ohio

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Portable masonry saw

A portable masonry saw for use on scaffolds, inside boilers, on the back of a truck, or on a work bench, has been introduced by Champion Manufacturing Co.

Called the "Tak-A-Bout Jr." by its manufacturer, the unit is available with a 1- or 1½-hp, fan-cooled motor. It can use abrasive or diamond blades; it features 14-in. blade guard capacity, a self-priming pump, a diamond lock, and a removable mandrel. Both wet and dry cutting models are available.—Champion Manufacturing Co., 3700 Forest Park Ave., St. Louis 8, Mo.

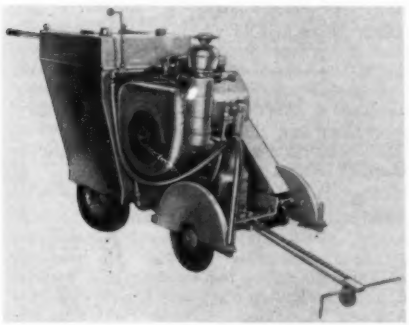
Quick splice rubber waterstop

Splicing time for the Gates Kwik-Seal rubber waterstop is just 6 min according to the manufacturer. Only a small splicing kit and simple clamping device are needed to provide a strong, permanent bond; no heat is required, no vulcanizers, no molded parts.

Manufacturer further states that Kwik-Seal rubber waterstop retains a watertight seal even when movement occurs in a concrete joint. It is recommended to prevent seepage and leakage wherever concrete is used: dams, power stations, bridge abutments, building foundation walls, swimming pools, etc.—E. J. Busch, Industrial Advertising, Gates Rubber Co., 999 South Broadway, Denver 17, Colo.

Power driven cutter

Engineered Equipment's Tri-Line concrete cutter saw is designed with a lockable third wheel for straight cutting. The unit operates



like a powerful rip saw and can be used effectively on concrete and asphalt according to manufacturer.

Other features of the cutter are tricycle undercarriage for maneuverability, variable speeds, and direct hydraulic blade depth control; powered by 2- and 4-cycle air-cooled gasoline engines; optional equipment are water tanks and water pumps.—Engineered Equipment, Inc., Waterloo, Ia.

Concrete masonry system for houses

A system of block manufacture and construction has been developed to facilitate construction and improve the appearance of a concrete masonry house.

The block are precision made on top, ends, and bottom, and are not cut but made in lengths of 4, 8, 12, and 16 in. The building

is laid out without a rule, the block being the measure. When the first row of block is laid level, the balance of the building should follow without requiring leveling. The solid color block have smooth surfaces with quarter-round edges. When laid in the wall they look like V-joints. In the center of each block, impressions are cast to exactly match the V-joints.

The block are laid in asphalt cement. The block layer applies the cement with a flat nozzle gun on a wall and the ends of the block which are put in a row. Then the asphalt cement ribbon is applied with one operation on end and wall. Woodwork is not used for frames, jambs, stops, cleats, etc. Openings are so precision-made that no fitting is said to be required. Hanging of doors, windows, etc., is directly to the block. The windows, fly screens, and shutters, slide in the double wall cavity horizontally and are used as required. They can be of wood or metal.

The vertical cores in the walls and horizontal ducts in the floors are used for heating and electrical distribution. Plumbing pipes are precut with most of the fittings fitted onto the pipes before they are placed in the horizontal ducts and vertical cores.

No plastering is required. The blocks are smooth and divided into squares with natural and imitation joints which give a tile appearance.—R. W. Stencel System, 5450 St. Joseph Blvd., Lachine, Montreal 32, Que., Canada.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

INTRODUCING CRETE-QUIP (Bulletin 72)—Literature explains the labor-saving features of Crete-Quip, a concrete hopper and bucket assembly for use with conventional materials hoisting towers. A large diagramatic illustration identifies the component parts of Crete-Quip, with photo sequence explaining the two-man assembly operation.

KWIK-SEAL WATERSTOP TECHNICAL MANUAL—Design manual for architects and industrial contractors contains 51 drawings and photographs. Covers sizes and shapes of the waterstop, comparative data for rubber and plastic waterstops, handling and storage of the material, design data for both "little" and "considerable" movement joints, application data for the installation of Kwik-Seal waterstop, and how to splice this material in 5 min.—The Gates Rubber Co., Industrial Division, 999 South Broadway, Denver, Colo.

A LOOK AT THE RECORD OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT—Performance report prepared by the Concrete Reinforcing Steel Institute committee on continuously reinforced concrete pavement. The 32-page digest covers activity in this field over the past 21 years, surveying projects in eight states. Discusses design, construction, soil foundation, maintenance and repair, and traffic conditions. Tables give summary of the survey.—Concrete Reinforcing Steel Institute, 38 S. Dearborn St., Chicago 3, Ill.

TWINWELD EPOXY TEST KIT—Manufacturers of TwinWeld epoxy adhesive are offering a free TwinWeld epoxy test kit. The sampling kit contains equal quantities of epoxy resin and hardener sufficient for an immediate "at the desk" test.—Fybrglas Industries, Division of Schramm Fiberglass Products, Inc., 3010 W. Montrose Ave., Chicago 18, Ill.

THORNTON FRACTIONAL HIGH SCHOOL (Flexicore Facts No. 87)—Folder tells how the use of precast concrete floor and roof decks with fireproofed steel columns and beams permitted contractors to continue construction of a $2-million high school in Lansing, Ill., through winter months. Detail drawings and photographs show how the Flexicore precast units facilitate all-weather erection. —The Flexicore Co., Inc., 1932 E. Monument Ave., Dayton, Ohio

NEW BEAUTY IN CONCRETE BLOCK—Booklet is aimed at increasing the use of concrete block, particularly the many new decorative and architectural designs. Interior and exterior uses of decorative block are illustrated. A section is devoted on how to build a garden wall and another section summarizes through illustrations step-by-step instructions for building with concrete block. The booklet will also be available for block manufacturers and dealers to offer potential customers—Permanente Cement Co., Kaiser Center, Oakland 12, Calif.

SOIL-CEMENT SHOULDERS FOR MODERN HIGHWAYS—Sound-color film describes the features of modern highways with special emphasis being placed on all-weather shoulders. The ease and speed of construction and strength gain with age of soil-cement are also stressed. Available on loan from the PCA Paving Bureau, or it may be purchased for $40.35, including reel, can, and shipping case. —Portland Cement Association, 33 West Grand Ave., Chicago, Ill.

PRECAST CONCRETE STRUCTURAL MEMBERS (Design and Data Sheet No. 8)—Four-page folder explains how precast T-beams, grade beams, rectangular beams and columns, and special beams and lintels can be engineered to fit the individual design and structural requirements of various type buildings, and how they have been used on specific jobs.—The Flexicore Co., Inc., 1932 East Monument Ave., Dayton, Ohio

THERMIT WELDING OF REINFORCING BARS (Information Sheet No. 5)—Information on Thermit welding of reinforcing steel for heavy concrete construction is given. Describes a new welding process developed specifically for this type of work; welding procedures are illustrated and explained in detail.—Thermex Metallurgical, Inc., Lakehurst, N.J.

SILICONE WATER REPELLENT FOR MASONRY SURFACES—Catalog on Dehydratine No. 22 silicone water repellent for masonry surfaces. Describes the product and method of application.—A. C. Horn Companies, Division of Sun Chemical Corp., 2133 85th St., North Bergen, N.J.

THE SHAPE OF THINGS TO COME—This issue of Perlite Design News describes the use of perlite insulating concrete for roof deck construction. Covers specific applications and problems that were overcome by the use of perlite insulating concrete.—Perlite Institute, Inc., 45 West 45th St., New York 36, New York.

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